PSEUDO NON-LINEAR SEISMIC ANALYSIS

by

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A method of evaluating the damage pattern and deflections of planar structures under earthquake excitation is presented. The proposed method is an iterative procedure based on ordinary elastic modal analysis, but it is extended to the inelastic range by using a special technique to take into account the reduction in effective stiffness and the variation in damping.

The theoretical development of the method is reviewed in this study and some improvements are also made. These new developments are then incorporated in an existing computer program for the method and tested with different kinds of idealized structures. Test results show good agreement with results obtained from a time-step analysis program DRAIN-2D, provided that the fundamental response period of the structure does not oscillate near the steep descending branch of the smooth spectrum and no extensive yielding occurs in the columns.

The proposed method is less expensive than an inelastic time-step analysis and can produce far more superior results than the elastic modal analysis; therefore it is considered as a good alternative to the two conventional types of analysis.
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1.0 INTRODUCTION

1.1 General Background

Although earthquakes of destructive intensity occur infrequently and only in certain areas of the world, adequate procedures to ensure safety of structures in those areas are important. But, designing earthquake-resistant structures is not a simple task. The difficulties in this area of study are inherent in the dynamic nature of the problem and the random variations in earthquake motion.

When a structure is subjected to severe earthquake excitation, some members of the structure will experience strains that go beyond their elastic capacity. As members respond in the inelastic range, the stiffness of the structure will decrease. The reduction of stiffness associated with the large-scale absorption of energy, in effect, changes the period of vibration and the damping of the structure during the course of the earthquake. With these variations, it is difficult to determine structural parameters, such as stiffness and damping, in a dynamic analysis. In the case of reinforced concrete structures, these variations are more pronounced, due to the change in sectional properties as concrete cracks during earthquake loading.
Another contribution which further complicates the matter but is often neglected when doing dynamic analysis stems from the properties of the soil and foundation. Soil-structure interaction is important, especially in the design of structures situated on relatively soft soils. Its effect on the response of the structure depends on the properties of the soil and that of the structure, as well as on the characteristics of the earthquake.

To predict the characteristics of future earthquakes to which a structure may be subjected in its lifetime is another uncertainty associated with the problem. First of all, earthquakes are generally random in character. Secondly, the magnitude of ground acceleration that a structure may experience depends on the magnitude of the earthquake itself and the distance of the structure from the epicenter. Further, it also depends on the properties of the surrounding soil and nearby constructions.

Fortunately, due to the advances in seismological study and the increasing number of seismographic stations, a large number of earthquake records are available for performing seismic analysis. And through studies of these records, estimates of the expected range of ground acceleration in a particular seismic zone are possible. However, because of the random nature of earthquake motion, the exact character of future earthquakes is still unpredictable. Records such as that of the 1940 El Centro event are still used widely in performing analyses despite the fact that they may not be suitable for the area of interest.
During the past decades, catastrophic failure of some buildings that went through major earthquakes did occur. But in studying these buildings and others that survived the earthquakes, more understanding of the actual behaviour of structures in earthquakes is gained. Observations combined with analytical studies of the response of some of these structures have proven that it is possible to predict the performance of a structure analytically to a certain degree of accuracy for a given earthquake.

Because of these developments, at least three kinds of seismic analysis, each suitable for different situations, have evolved and are commonly practised in the analysis and design of earthquake-resistant structures. The following is an overview of these methods.

Of the three methods, the simplest one is called the equivalent lateral force procedure. It is suitable for designing structures with regular distribution of mass and stiffness over height, or structures that are not located in highly seismic areas. It is easy to use because it only requires the value of the fundamental period of vibration, which can be estimated by using a simple formula, and an estimation of the damping. Based on these data, with an appropriate design spectrum, derived from the observed behaviour of buildings in earthquakes, a set of equivalent lateral forces is generated. With this set of equivalent lateral forces, the remaining procedure is no more than an ordinary static analysis. Therefore, this method is especially useful for the purpose of doing preliminary
A more rigorous analytical method is 'modal analysis'. In contrast to the equivalent force method, the periods and the mode shapes of at least the first few modes of vibration are needed. The magnitude of the lateral forces for each mode are then determined from the response spectrum. The root-sum-square of the response for each vibration mode is taken as the envelope, which represents the maximum response the structure would exhibit during the earthquake, if it were to remain elastic. It has been observed by Newmark and Hall that the displacement response of a single degree of freedom system is essentially the same, whether or not it yields. The geometry of the force-deformation diagram then shows that, if the response is elastic-perfectly plastic, the yield force will be the force obtained from the elastic analysis divided by the ductility. (Ductility is defined as total displacement divided by yield displacement.) It is assumed, therefore, that the modal analysis just described gives approximately correct displacements, and that yield-level forces for design purposes can be obtained by dividing those found in the analysis by the available ductility inherent in the structural system. Clearly, these assumption are invalid for the multi-degree-of-freedom system unless it undergoes uniformly dispersed yielding, which will only occur if the distributions of stiffness, mass, and strength are also uniform.

The two preceding methods are simple to apply, but neither is applicable if the structure is not simple,
uniform, and 'well-behaved'. Therefore when dealing with larger or more complex structures, the more sophisticated inelastic time-step analysis is required. Time-step analysis is based on a different concept from the two previous methods. An earthquake record, instead of a response spectrum, is used to determine the response of the structure at any instant during the earthquake. The energy dissipating mechanism of the system and the non-linear characteristics of structural elements are more accurately represented by modelling the hysteresis effects. This analysis is, thus far, the most reliable available method of dynamic analysis. It not only yields the maximum response of the structure, and the approximate time at which this maximum occurs, but also makes the whole response time history available for examination.

However, the complexity of time-step analysis makes it impossible to use without the aid of a large computer; and running such a program is both expensive and time-consuming. The results obtained from such an analysis are only valid for one set of structural properties and one earthquake record. Changes in structural properties, which frequently occur as a design proceeds, mean additional runs of the program and additional cost to the designer. Sometimes, in order to gain thorough understanding of the behaviour of the designed structure, more than one input earthquake record is used. These factors make the analysis so expensive that it is generally out of the reach of average engineering practice and only used in large projects where enough funds
can be justified.

In view of these comments, the hope of obtaining better and more reliable dynamic analysis appears to rest on several different factors. One possibility of course, is that rapid development in the field of computer technology will lead to more powerful computers, which will lower the cost of executing complex analyses. In that case, the inelastic time-step analysis appears to be the ultimate answer. However, it will still be a long time before the cost of preparing and running such programs can be brought down to a level that is attainable for the average engineering office for small to medium buildings. Therefore, it is desirable to improve modal analysis so that solutions can represent more adequately the response of structures in the inelastic range.

In order to make modal analysis work in the inelastic range, attempts have been made to develop an inelastic response spectrum from the elastic response spectrum. In this method the ordinary modal analysis can be performed in the usual way, but the inelastic spectrum is used to determine the spectral acceleration for each mode. The inelastic response spectrum is derived from the elastic spectrum with an allowable ductility factor, which is determined from the structural form and the properties of the materials used. Unfortunately, it suffers from the same shortcoming as the equivalent lateral force procedure, and can only be applied to structures where ductility demand is expected to be distributed uniformly.
A relatively simple method, also based on modal analysis, which is called the substitute structure method, was introduced by Shibata and Sozen\textsuperscript{2}. This method provides a simple means of taking into account the inelastic properties of the structure. It is a design procedure whereby the designer can assign different limits of inelastic response to each member. Then different values of damping and stiffness for all the members are calculated and modified by the corresponding predefined limits, in such a way as to give equivalent linear members with viscous damping representing the hysteretic damping of the real non-linear members. One equation is also provided to determine the system damping from values of member damping. Finally, the design yield forces of the members are obtained as the results of an ordinary modal analysis.

The present method is an analysis based on the same theory but with the above procedures reversed. In other words, the design capacities are predefined and the inelastic limits are treated as unknowns. Since the stiffness and damping of the structure are also functions of these limits, an iterative procedure must be used. Details and developments of this method are discussed in Ref. 3, 4 and also in the following chapters. It is proposed as a method of identifying anomalous behaviour in structures which have been designed by an equivalent lateral force procedure.
1.2 Objective Of Study

The three methods of seismic analysis commonly used in practice were introduced in the last section. It was pointed out that although the equivalent lateral force procedure or modal analysis can provide simple means of performing dynamic analyses, both methods are restricted by their inability to predict accurately the inelastic behaviour of structures. Inelastic time-step analysis is well-known to be the most powerful and informative method in the area of seismic analysis. However, the application of such procedures is limited by the high cost involved in both data preparation and execution.

The present method which was developed in recent years, is intended to fill the gap between the relatively simple elastic modal analysis and the more complicated inelastic time-step analysis. The advantages of the proposed method are:

1. The inelastic response of a structure can be determined by a linear modal analysis using an elastic response spectrum;
2. The reliability of solutions obtained is comparable to inelastic time-step analysis; and
3. The execution cost of such a program is substantially lower than that of the time-step analysis.

These advantages were verified through this study.

However, the proposed method was also subject to a
number of limitations, which originally applied to the substitute structure method suggested by Shibata and Sozen\textsuperscript{2}. They impose the following requirements on the structure:

1. The system can be analysed in one vertical plane.
2. There are no abrupt changes in geometry or mass along the height of the system.
3. The limits of inelastic response for columns, beams and walls can be different, but these limits should be the same for all beams in a given bay and all columns on a given vertical axis.
4. The force response should decrease as the structure becomes more flexible.
5. All structural elements and joints are reinforced to avoid significant strength decay as a result of repeated reversals of the anticipated inelastic displacements.

Since the discussion in this thesis is restricted to two dimensional analysis, the first limitation still applies. Limitation (4) can be overcame by the method suggested by Shibata and Sozen\textsuperscript{2}; that is, if in the range of the lower modes of the structure the spectral acceleration response increases with an increase in period, a constant acceleration response, up to the period at which the response starts decreasing, can be assumed. Limitations (5) and (6) apply to most dynamic analyses, even in time-step methods. However, no explanation was given for the limitations stated in (2), and (3) even in the original
paper of Shibata and Sozen; therefore, one objective of this study is to show when restrictions (2) and (3) are unnecessary, at least in the present method.

The program for the proposed method was first developed by S. Yoshida in 1979. At that time, a variety of small frames were tested using the first version of the program and results were presented in Ref. 3. Subsequently, the program was modified by A.W.F. Metten to extend its capability to include the analysis of structural walls. Again, a number of different coupled shear-wall structures were examined and results were presented in Ref. 4. As an extension to the previous studies, the objective of this study includes further improvement to the program. Areas to be improved include:

1. study and resolve problems that hinder proper convergence of the program during the course of this study;
2. investigate the adequacy of the element damping formula; and
3. remove simplifications that were imposed on the proposed method in previous studies.

It was mentioned in the last section that modal analysis is most suitable for analysing medium size structures; therefore it is important to know whether the proposed method is adequate for analysis of such systems. A limited number of parametric studies will be conducted to investigate the effect of varying certain parameters on the reliability of the method. These results will be compared to
a standard inelastic time-step analysis.

1.3 Scope

Since a detailed description of the present method has been provided in each of the previous studies (Ref. 3 and 4), only a brief summary of the method is presented in this thesis. Readers are advised to refer to one of the above references when more details of the method are required. However, a deeper look into the structure of the program is provided here in the hope of aiding further improvement to the existing program in future studies.

In the beginning of this study, a number of structures analysed by the modified version of the program revealed some convergence problems. Careful examination of these problems showed that there are other limitations to the method that were not realized in the previous studies. These convergence problems and the limitations that they imposed on the method, as well as a proposed routine to ensure convergence, are discussed in chapter 3.

During the course of studying the above topic, the adequacy of the member damping formula was also investigated. The results are presented in Chapter 4.

In previous studies, a number of simplifications were included in the proposed method. In this study, these simplifications were removed in order to increase the applicability and reliability of the method. These improvements are discussed in detail in Chapter 5.
Finally, no study is completed without performing tests on the program. A number of medium size structures was designed and each of them was specifically used to test certain functions of the program. The results obtained are compared with a standard inelastic time-step analysis. To prove that improvements were obtained through the above mentioned variations, a comparison of results with the previous program was also made. These results are presented in chapter 6, along with a final note on the execution cost of the program.
2.0 THE PRESENT METHOD

2.1 Brief Review Of The Present Method

The present method is an iterative procedure for analysing the response of structures under earthquake excitation. The method was developed from the substitute structure method, which is a design procedure based on modal analysis, proposed by Shibata and Sozen. In order to understand the modified method, it is easiest to start with the original substitute structure method; the following is an introduction to that procedure.

2.1.1 The Substitute Structure Method

The basic ideas of the substitute structure method are as follows:

1. The acceptable limit of the inelastic response for each member is established according to the materials and the desired level of detailing.

2. The reduced stiffness of an equivalent linear member which would respond in a similar way is calculated.

3. A fictitious level of viscous damping which dissipates an amount of energy equivalent to that lost in the hysteretic loop of the real inelastic member is deduced.
4. This fictitious damping for each of the members is combined into a structure damping for each mode.

5. A modal analysis is made for the equivalent linear structure with the reduced stiffnesses and the calculated viscous damping substituting for hysteretic effects in the real system.

In modifying the stiffness, the 'damage ratio' is used; this quantity is rather similar to the member ductility ratio. It is important to distinguish damage ratio from member ductility ratio which is more commonly used in practice. Member ductility ratio, which will be referred to as the ductility ratio from here on, is defined as the ratio of ultimate displacement to yield displacement. This concept can be illustrated by the example shown in Fig. 2.1. The figure shows a typical frame member displaced from its original position in Fig. 2.1(a) to the position at Fig. 2.1(b). Assuming that the end moments are equal and opposite, the moment diagram of the member will be as in Fig. 2.1(c). Fig. 2.1(d) is the moment vs. rotation diagram for the two ends of the member. In this case, the ductility of the member is simply

\[ \eta = \frac{\theta}{\theta_y} \] (2.1)

where \( \eta \) is the ductility ratio

\( \theta \) is the end rotation of the member defined by the angle shown in Fig. 2.1(a).
On the other hand, damage ratio is defined as the ratio of the initial stiffness of the element to the secant stiffness of the element in its final configuration. This secant stiffness is the stiffness for the substitute structure.

Using the same illustration in Fig. 2.1(d), the slope of line OA is the initial stiffness of the element, slope of line OB is the apparent stiffness of the element which is used in the substitute structure method. The damage ratio is

\[ \mu = \frac{\text{slope of OA}}{\text{slope of OB}} \]  \hspace{1cm} (2.2)

Although ductility ratio and damage ratio are defined differently, they are related by the following equation,

\[ \mu = \frac{\eta}{1 + (\eta - 1) s} \]  \hspace{1cm} (2.3)

where \( s \) is the strain hardening ratio in proportion to the initial stiffness.

Equation (2.3), shows that the ductility ratio and damage ratio are equal only when strain hardening ratio is zero, which is true for elasto-plastic response. In any other situation, the damage ratio would be smaller than the ductility ratio.

After the damage ratios are assigned to all the elements of the structure, the next step is to obtain the stiffness of the substitute structure:
EI\textsubscript{s,i} = \frac{EI\textsubscript{a,i}}{\mu_i} \quad (2.4)

where \(EI\textsubscript{a,i}\) is the sectional stiffness of the element in the actual structure
\(EI\textsubscript{s,i}\) is the sectional stiffness of the element in the substitute structure
\(\mu_i\) is the damage ratio assigned to element \(i\).

In order to perform the modal analysis, an estimate of the damping factor for each mode is needed. An equation to calculate damping for concrete elements was developed just for this purpose by Gulkan and Sozen\textsuperscript{5}. The substitute damping is approximated by the following equation,

\[ \beta_i = 0.02 + 0.2 \left( 1 - \frac{1}{\sqrt{\mu_i}} \right) \quad (2.5) \]

where \(\beta_i\) is the substitute damping factor for element \(i\).

To obtain a damping factor for the entire structure, the individual values of \(\beta_i\) have to be combined by applying some kind of averaging technique. In the substitute structure method, the structural damping is obtained from the weighted average of all the element damping factors, and the weight factor used is the strain energy generated from each element. This strain energy is calculated as follows.
\[ \Pi_i = \frac{L}{6(EI_{si})} (M^2_{Ai} + M^2_{Bi} - M_{Ai} M_{Bi}) \] (2.6)

where \( \Pi_i \) is the strain energy of element i

\( EI_{si} \) is obtained from Eqn. (2.4)

and \( M_{Ai}, M_{Bi} \) are end moments of element i

( refer to Fig. 2.1(b) )

and thus the structural damping factor is,

\[ \beta_s = \frac{\sum (\Pi_i \beta_i)}{\sum \Pi_i} \] (2.7)

where \( \beta_s \) is the 'smeared' damping factor for the substitute structure.

To simplify the process of picking the spectral acceleration for a given damping factor from a smooth response spectrum, which would usually show curves corresponding to certain selected damping values only, the following expression is used:

\[ S_{a_s} = S_a (@ 2\% \text{ damping}) \times \frac{8}{(6 + 100 \beta_s)} \] (2.8)

where \( S_{a_s} \) is the spectral acceleration, derived from the value at 2\% damping.

To start the procedure, first, the stiffness matrix and the mass matrix are assembled. The modal periods and mode shapes are determined using any standard Eigen-value solver. Since the equation for calculating damping (Eqn. (2.6)) involves the end moments, which are the required quantities,
an initial estimation of the smeared damping factor for each mode is required in order to determine the spectral acceleration from the response spectrum. But, alternatively, an arbitrary value of spectral acceleration can be assumed. Then the modal analysis can be performed as usual which yields a set of resulting end moments. With these results, it is possible to refine the values of smeared damping using Eqn. (2.6) and (2.7) for each mode. After that, a new value of spectral acceleration can be determined for each mode and by repeating the modal analysis the final design moments and displacements are obtained. An iterative procedure is not needed because the magnitudes of the end moments used in Eqn. (2.6) can be pro-rated for different values of spectral acceleration, due to the fact that the stiffnesses of the members are fixed during this process.

2.1.2 Proposed Modifications Of The Substitute Structure Method

The modified substitute structure method is an analysis, which means that all structural properties, such as initial stiffness and member capacities, are known quantities. In other words, moment-rotation diagrams such as that in Fig. 2.1(c) are defined for each member. What is not known is the status of the members at maximum response of the structure in a given earthquake motion. In this method, the status of each member will, again, be defined by the damage ratio as in the substitute structure method. However, the member responses are known, and the damage ratios are
the required quantities, whereas, in the original proposal of Shibata and Sozen\textsuperscript{2}, the tolerable damage ratios are known and the yield values of the member forces are the required quantities. Since they are not known, a first guess at these damage ratios is required. With these trial values, the procedure in the substitute structure method can be performed exactly as described in the last section. The end result will be a set of end moments, axial forces, shear forces and displacements; the end moments of each member must conform to its own capacity. If the end moment of a member is different from its capacity, it simply means that the trial value of the damage ratio is incorrect and must be adjusted. An improved trial value can be obtained from,

\[ \mu_n = \mu_{n-1} \frac{M_n}{M_Y} \geq 1 \]  \hspace{1cm} (2.9)

where \( \mu_n \) is the new damage ratio
\( \mu_{n-1} \) is the damage ratio from the last calculation
\( M_n \) is the larger of the two end moments of a member
and \( M_Y \) is the yield moment of the member.

With strain hardening, the expression would be

\[ \mu_n = \frac{M_n \mu_{n-1}}{M_Y (1 - s) + s \mu_{n-1} M_n} \geq 1 \]  \hspace{1cm} (2.10)

where \( s \), again, is the strain hardening ratio as defined previously.
Inevitably, this is an iterative procedure where the damage ratios have to be refined through each iteration until moments on every member match the moment capacity. In refining the trial value of the damage ratios, it must be noted that these trial values cannot be less than one in beginning of any iteration because the stiffness of the substitute structure is not allowed to be larger than that of the actual structure (refer to Eqn. (2.4)) in any situation.

Although it is theoretically possible to obtain a set of damage ratios for which the moments match exactly with the capacity of the members, it is practically unfeasible because of the vast number of iterations required. The accuracy of the results is also limited by other factors. Therefore, somewhat relaxed criteria are employed. These criteria will be introduced in the following chapter. In general, a relative tolerance between resulting moment and member capacity of less than five percent will produce reasonable results. For the criteria adopted in the program, convergence is usually attained in fewer than ten iterations, which is significant for computational economy.

2.2 The Development Of The Computer Program

A program for executing the suggested procedure can be obtained easily by modifying an elastic modal analysis program. One way to accomplish this is to incorporate
Eqns. (2.4) through (2.8) into any elastic modal analysis program so that it can perform the substitute structure method. After that is done, to introduce the necessary modification, a loop must be inserted into the program to make the procedure an iterative one. Equation (2.9) or (2.10) must be included in the program so that the damage ratios can be modified in each iteration. Then the final step is to impose a convergence scheme in order that the program can be stopped once the solution has reached the required limit of accuracy. These are essentially all the ingredients that are contained in the first version of the program that was developed by S. Yoshida.

The second version, edited by A.W.F. Metten, came as a refinement to the first version of the program. A number of features was added to the program to make it more efficient. One of these features is the 'convergence speeding routine' which speeds up convergence by improving the trial value of the damage ratios after each iteration based on some extrapolation technique. However, this is effective only after the eighth iteration is performed. Another feature is the relaxation of the convergence limits that eliminates unnecessary iterations which do not improve the accuracy of the solution. The second version of the program also allows for structural members with rigid ends. This feature opened the possibility of modelling structural walls.

The latest version of the program is, again, a refinement to the previous one. Improvements made in this version were already mentioned briefly in section 1.2 and
will be discussed in detail in the following chapters as well.

2.3 Introduction To The Structure Of The Computer Program

The intention of this section is to explain the basic structure of the computer program so that further improvement to the program can be made with ease. The program to be introduced in the following paragraphs will have the structure of the third version but the capability of the second version only; the additional features for the third version will be added onto the flowchart after they have been introduced in next few chapters. One major change in structure of the second version is in the main subroutine MOD3. MOD3, which is a ten and a half pages long subroutine in the second version, has been shortened to half of its original length. The functions of the remaining half are shared by two new subroutines, FORCE and DAMOD. It is believed this change will make the program much easier to understand and also easier to modify in future.

In Fig. 2.2, a flowchart of the main program is shown. This flowchart follows the same sequence in which subroutines and instructions are executed in the program.

The first subroutine CONTRL is used to read in non-structural information, e.g. number of modes to be analysed, maximum ground acceleration, initial damping etc., and structural information that is common to the whole
structure, e.g. the Young's modulus, strain hardening ratio etc. The second subroutine SETUP is also for input purposes where structural information such as joint coordinates, member size, sectional stiffnesses, and moment capacities are read in. These data are reorganized within the subroutine and the half bandwidth of the matrix is determined. The damage ratios are also initialized to one in the subroutine, to provide the first set of trial values. Data input into the program through these two subroutines are all echo printed so that the user can check whether information is being read in correctly. In the next step, the mass matrix is assembled in the subroutine MASS. Since this matrix does not vary throughout the iterative procedure, it is only required to be calculated once. After all this information is returned to the main program, the next step is to define some control variables such as convergence limits, iteration number etc. These finalize the preparation stage of the program and the following step is to proceed to the iteration process.

In the beginning of each iteration, subroutine BUILD is called to assemble the stiffness matrix of the substitute structure. This will involve the use of equation 2.4, in which the stiffness of the substitute structure is defined as a function of damage ratio. After that, the program will check for ill-conditioning of the stiffness matrix using subroutine SCHECK. The ratio of largest to smallest diagonal elements is calculated and printed out from the subroutine. However, it is up to the user to determine whether the ratio
is acceptable or not. If the stiffness matrix does not contain diagonal elements smaller than zero, the program will proceed to the next subroutine EIGEN. The function of subroutine EIGEN is to determine the modal periods and mode shapes. This is done through the Eigen-value solver SPRIT which is called inside EIGEN. After EIGEN comes the main subroutine of the program, MOD3, which actually performs the modal analysis to come up with the resulting forces, displacements and damage ratios. It also prepares the results for the convergence check. When this information is returned, instructions in the main program will check whether convergence has been acquired. If the check is positive, the program will stop after performing one more iteration and printing results at the same time. Otherwise, a second iteration will be performed again starting with subroutine BUILD, where the stiffness matrix will be modified by the new set of damage ratios.

Since subroutine MOD3 is the heart of the computer program, it deserves further explanation. As a reminder, MOD3 is executed after the modal periods and the mode shapes have been found.

Fig. 2.3 is a flowchart of the subroutine MOD3. The subroutine starts out with the determination of the modal participation factors for each mode. Then the procedure, which is enclosed by dotted lines in Fig. 2.3, is repeated twice for all modes within the subroutine. The function of this procedure is to compute the resulting forces and displacements. In the first time through the procedure, the
end moments obtained will be used to calculate the smeared damping factors using Eqns. (2.5), (2.6) and (2.7). In the second time, the actual response of the structure is determined base on these damping factors. This is done exactly as described in section 2.1.1.

Following the procedure in Fig. 2.3, first, the spectral acceleration is determined in subroutine SPECTR. Equation (2.8) is applied to scale the spectrum according to the damping factors that are obtained from the last iteration. With the spectral acceleration known, modal forces applied to the structure can be calculated. Next, the matrix subroutines SDFBAN and DSBAND are used to compute the modal displacements from the already assembled stiffness matrix and modal force matrix. The displacements of each mode are then squared and summed up. After that, a subroutine FORCE is called to determine the resulting forces base on those modal displacements. If this is the first time through the loop, i.e. when KK is set equal to one, the smeared damping factor for that mode will also be calculated. Otherwise, the program will move on to calculate the root-sum-square displacements, and then go directly to the next subroutine DAMOD. In DAMOD the resulting end moments are used to modify the damage ratios according to Eqn. (2.9) or (2.10). It will also calculate the relative difference in damage ratios between current and last iteration in preparing for the convergence check in the main program. Furthermore, the convergence speeding routine that was mentioned in the last section is also contained in this
subroutine. Details of the convergence speeding routine are discussed elsewhere, in Ref. 4. Finally, the information from MOD3 is returned to the main program, and the convergence check will be carried out immediately thereafter.
3.0 CONVERGENCE

3.1 Convergence Schemes

Since the proposed method is an iterative procedure, some convergence criteria must be imposed to halt the process when an acceptable level of accuracy is acquired. Basically, this has been done by previous workers through the following criteria (Ref. 3 and 4)

\[
\left| \frac{M_n - M_{\text{cap}}}{M_{\text{cap}}} \right| < 0.05 \quad \text{for } \mu_n > 1 \quad (3.1)
\]

and

\[
\left| \mu_n - \mu_{n-1} \right| < 0.1 \quad \text{for } 1 < \mu_n < 5 \quad (3.2a)
\]

or

\[
\left| \frac{\mu_n - \mu_{n-1}}{\mu_n} \right| < 0.01 \quad \text{for } \mu_n \geq 5 \quad (3.2b)
\]

Regarding these criteria, there are a few points worth noting. First of all, Eqn (3.1) is used to ensure that the moment of each yielded member would conform to its capacity. The number of members that have end moments deviating from their capacity by over 5 percent will be counted and then printed out under the heading 'no. above capacity' after
each iteration. This is sometimes quite misleading, because the use of the absolute sign tends to over-count the number of members by including those which are temporarily below capacity but with damage ratio larger than one. This situation frequently occurs when the spectral acceleration drops in the second iteration and when a convergence problem is encountered.

Before making any adjustment, it may be worthwhile first to understand some implications of these criteria. Since Eqn. (2.9) relates moments to damage ratios, therefore from

\[ \mu_n = \mu_{n-1} \frac{M_n}{M_Y} \] (2.9)

Eqn. (3.1) can be rewritten as

\[ \frac{M_n - M_{cap}}{M_{cap}} = \frac{\mu_n - \mu_{n-1}}{\mu_{n-1}} \] (3.3)

If \( \mu_{n-1} \) is equal to two, then obviously criterion (3.1) is the same as criterion (3.2a). In another word, it means that criterion (3.1), in fact, will govern only when \( \mu_{n-1} \) is less than two. With these in mind, the aforementioned criteria can be redefined as follows:

When damage ratio is increasing,

\[ \frac{M_n - M_{cap}}{M_{cap}} = \frac{\mu_n - \mu_{n-1}}{\mu_{n-1}} < 0.05 \] for \( \mu_n > 1 \) and \( \mu_{n-1} \leq 2 \) (3.4)
\[ |\mu_n - \mu_{n-1}| < 0.1 \quad \text{for } 1 < \mu_n < 5 \text{ and } \mu_{n-1} \geq 2 \quad (3.5a) \]

\[ \left| \frac{\mu_n - \mu_{n-1}}{\mu_n} \right| < 0.01 \quad \text{for } \mu_n \geq 5 \quad (3.5b) \]

When damage ratio is decreasing,

\[ |\mu_n - \mu_{n-1}| < 0.1 \quad \text{for } 1 < \mu_n < 5 \quad (3.5a) \]

\[ \left| \frac{\mu_n - \mu_{n-1}}{\mu_n} \right| < 0.01 \quad \text{for } \mu_n \geq 5 \quad (3.5b) \]

From the conditions of the above criteria, it is shown that criteria (3.5a and b) covered the largest range of damage ratios. Since tests conducted throughout this study involved large damage ratios, the results also indicated that these two criteria governed convergence of the program all the time. Therefore, criterion (3.4) would only control the process if most of the damage ratios are below two. Also because criterion (3.4) is much tougher than (3.5a), dropping the absolute sign would tend to give more conservative results under certain situations, which in fact is desirable when damage ratios are close to one. So, in this version of the program, the absolute sign is dropped. It can be shown later on, that these criteria can be used as indicators of progress towards convergence as well.

Another point concerning these criteria is, as A.W.F. Metten had pointed out in his thesis (Ref. 4), the
main reason for adopting those limits was that the damage ratios calculated by using this method is only significant to the first decimal digit. Test results in this study have further proven that this argument is in fact correct. Although it is believed that even greater relaxation to the above limits is still possible, the amount of saving resulting from doing so is insignificant.

As a last comment to this section, the above criteria provide only one of many ways to determine when the program should stop. Other alternatives are possible; one example is to use criterion (3.2b) alone and set up a counter, that counts the number of members which does not satisfy this criterion, to indicate the progress of convergence. However, it is important that the convergence limit to any criterion must be appropriate so that reliable results can be produced through the iterative method.

3.2 Effect Of Including Strain Hardening

As a strain hardening ratio is included in the program, criterion (3.4) must be altered to incorporate the increase in material strength due to this factor. This is done by raising the moment capacity using the following equation

\[ M_{cap} = \frac{M_y (1 - s)}{1 - s \mu} \]  

(3.7)

where \( M_y \) is the input moment capacity
s is the strain hardening ratio in proportion to the initial stiffness and \( \mu \) is the damage ratio.

This equation was derived from the same moment-rotation diagram used in deriving the damage ratio modification equation (Eqn. (2.10)), therefore the results should be consistent.

3.3 Factors That Hinder Convergence

Almost any analytical method has its weaknesses and limitations, and the present method is no exception. In the early stage of this study, numerous convergence problems were encountered. In order to find out the origin of these convergence problems, the program was altered so that variables such as the period, spectral acceleration, smeared damping, number of members above capacity (according to criterion (3.4)) and maximum variation in damage ratio (according to criterion (3.5a and b) for each iteration would be printed out to the terminal while the program was running. By doing so, the whole process, was put under the supervision of the user. The user was able to determine whether proper convergence was acquired during the process. In case of any convergence problems, the program could be stopped immediately from the terminal to prevent further loss in computer money.
With this additional aid, it was found that there were generally two cases that would cause trouble in convergence. These cases and the method of detecting them are discussed in the following paragraphs.

In the beginning of this study, it was intended to test structures that were supposed to be poorly designed. So an 8-bay, 7-storey frame was designed using ordinary elastic modal analysis. The moment capacity for all members was calculated based on the results of the analysis divided by a ductility of four. Obviously, a ductility equal to four for every member is a violation of present design philosophy. In practice, the ductility given to beams is usually much greater than to columns, so that yielding will occur in the beams rather than in the columns, thus providing the essential stability that is required to survive an earthquake. However, this structure might represent some of the older buildings that were constructed under the old design philosophy. So it was desired to see how the method would predict the response of such a structure.

When the program for this structure was running, the number of members above capacity stayed constant, and the maximum change in damage ratio also stayed above the required limits for a large number of iterations. However, the program did manage to converge after more than 20 iterations. The results indicated that most of the members were not damaged, except that the columns of one floor had damage ratios all above ten. To verify the results, a reputable time-step analysis was used to analyse the same
structure. Unfortunately, the results obtained from this analysis proved that the previous results were incorrect.

Since then, two more structures, including one shear-wall frame and one other frame similar to the first one, were designed using the same idea and tested using the two programs. The shear-wall frame converged within ten iterations and showed good comparison with the time-step analysis, except for a few members, but the same problem was encountered for the second frame. So, it was suspected that the proposed method would have a tendency to indicate a soft storey for structures that consist mainly of strong beams and weak columns. The shear-wall frame is an exception, probably because the high moment capacity of the shear-wall prevented the soft storey from occurring.

This kind of drawback, would certainly affect the usefulness of the method for 'retrofit' purposes. But the method is still a powerful analytical tool to analyse structures that are designed according to the present code. Besides, it is also arguable whether the performance of structures, which cause this kind of convergence problem, would be satisfactory under earthquake motion. Therefore, further work on this problem is required.

Another factor that would cause trouble in convergence is related to the input response spectrum. Almost all smoothed earthquake response spectrums have a similar shape (see Fig.3.1); they start with an ascending branch at low period (spectral acceleration increases with period); a constant branch follows, where there is no change in
spectral acceleration over a short range of period; then there is a descending branch, where spectral acceleration decreases with increasing period. These branches reflect the forces induced in a structure of a given vibration period.

The inelastic response period of structures varies widely depending on the construction materials used and the height of the structure, as well as the type of structural system considered. This response period also varies in the course of an earthquake. However, it is believed that there exists one unique vibration period, in a given earthquake excitation, that would yield the maximum response that causes the largest damage. In the substitute structure method, the response period for each mode is a variable that changes from iteration to iteration. In normal convergence, this period will increase, or decrease, at a diminishing rate, as results approach the final solution after each iteration. This kind of convergence is usually attainable provided that the spectral acceleration does not vary significantly in magnitude during successive iterations. Unfortunately, this variation is unavoidable once the iteration period falls into the rapidly descending branch of the response spectrum, which usually occurs at around the period of 0.5 sec.

In an attempt to explain the problem, suppose there is an abrupt reduction in spectral acceleration due to a slight increase in period during a subsequent iteration. The immediate effect of this will be a large decrease in damage ratios that leads to lower damping and vibration period.
This is reflected from the terminal output as a sudden decrease in the number of members above capacity and a negative difference in damage ratios. Because of the shortened vibration period, the spectral acceleration in the following iteration will return to a higher level. Such a variation will reverse the process of the last iteration, and return to higher damping and period again. From the terminal output, it will show an increase in the number of members above capacity and a positive difference in damage ratio. This mechanism will go on for a large number of iterations before the program finally converges.

Fortunately, this problem rarely occurs. When it does occur, it is likely that either the structure is small, or the structure is rather stiff so that its inelastic period falls into the vicinity of the rapidly descending branch. Fig. 3.2 is one example of the latter case. In this case the program only went through fifteen iterations to converge. Fig. 3.3 is the same example plotted with the response spectrum. The reason that the results seem to be converging to a point below the spectrum is that the damping has increased above 2 percent, and the spectrum has been modified thereby to a lower curve, not shown. One interesting point to note here is the effect of the convergence speeding routine on the problem of convergence. The figure shows that after the convergence speeding routine became effective in the ninth iteration, the oscillation pattern was broken, and the results converged. However, by observing the results of the example, it was found that real
convergence is actually not acquired, but rather, the results have satisfied the convergence criteria temporarily; in the last iteration, the error begins to increase again. It was also found that the convergence speeding routine does not necessarily bring about rapid convergence; in some cases it increased the magnitude of fluctuation, which means that it might create more problems than it solves. This kind of false convergence is less likely to be obtained without the convergence speeding routine, but a vast number of iterations will, of course, be required in such a case. A different and better procedure is described in the remainder of this chapter.

3.4 The Solution Searching Routine

From past experience, it was observed that even for cases where smooth convergence was acquired, there are generally some fluctuations in response period in the first four to five iterations, after which the period starts to approach smoothly to the final response period. But in the cases with convergence problems, the fluctuation will carry on after the fifth iteration. This characteristic provides an important clue for detection of convergence problems and indicates when the search routine should put into effect.

A concept that has helped the development of the search routine came from an analytical procedure proposed by S.A. Freeman. Although Freeman's method is a graphical one,
the basic theory behind it is actually the same as the modified substitute structure method. With Fig. 3.4, it is possible to explain some fundamental ideas behind his method. Fig. 3.4 is a plot of spectral acceleration versus response period. Line IJK represents the capacity of the structure, called the 'capacity' curve. For spectral acceleration up to joint J, the structure remains elastic, therefore the period is just equal to the initial elastic period. With spectral acceleration increased after point J, the structure begins to yield and the period becomes longer. This is reflected by the line JK. Since the structure has entered into its inelastic range, damping of the structure starts to increase as well. As a result, a transition occurs between the response spectrum corresponding to the initial damping of the structure and spectrum corresponding to higher damping values. The shape of this transition curve, called the 'demand' curve, will depend on the change in damping of the structure associated with the change in response period and also on the shape of the response spectrum itself. According to Freeman's method, the intersection between the capacity curve and the demand curve shows the maximum response of the structure for a particular earthquake.

This supports the previous contention that a unique solution exists. What the search routine should do is search for the intersection point along the capacity line once the direct iterative procedure has failed. (Note that Freeman's method does not indicate the distribution of damage
throughout the structure; the present method is intended to do this.)

One advantage of the existing program is that it is economical for use in special studies. Prior to the writing of the search routine, a special study on Freeman's method was conducted. It was done through the same example that was used in Fig. 3.3 and 3.4. In Fig. 3.5, results of the first mode from iteration 4 and 5 are plotted in terms of spectral acceleration and vibration period. The spectrum corresponding to the damping of each iteration is also plotted. Then the program was altered so that a constant first mode spectral acceleration can be fed in to be used instead of that read from the spectrum. So the spectral acceleration at 2 percent damping for the fourth and fifth iteration was calculated and then fed into the modified program. In the former case, the program converged to a point at A and in the latter case, the program converged to another point at B. During convergence, the damping changed to 5.2 percent at A and 6.6 percent at B. (Note that in iterations 4 and 5 above the damping was 5.3 percent and 6.4 percent respectively.) It is interesting to observe the results corresponding from other values of constant spectral acceleration. These results are plotted in Fig. 3.6 along with the spectrum corresponding to the final smeared damping factor for each case. Points 2 and 4 are the same as A and B, respectively, in Fig. 3.5. According to Freeman's method, the capacity curve is simply a line which goes through these points. The demand curve can also be obtained by joining
points on the spectrum lines that correspond to the final periods and smeared damping values of each of these sub-iterations. Then the solution suggested by Freeman's method would be the intersection of the two curves, i.e. 0.579 sec. and spectral acceleration of 0.292g.

It is interesting to find from Fig. 3.6 and Fig. 3.7 that the smeared damping tends to vary linearly over the periods while the transition of the spectrum does not. However, the most important point of this exercise is to show that the solution of the problem is actually bounded by the values of spectral acceleration corresponding to the early stages of the iterative procedure, like that for the fourth and fifth iteration. The significance of this is that these values of spectral acceleration can be used as the upper and lower bounds that will limit the range of the capacity curve where the solution has to be found. (Note that the validity of the upper and lower bounds can easily be checked, if necessary, since the upper bound will give a solution below the actual spectrum whereas the lower bound will give a solution above the actual spectrum, see Fig. 3.6.) Because of this, the following procedure can be used to set up a search routine:

1. To detect a convergence problem: after the first five iterations, start to compare the changes in first mode response period with the previous two iterations and determine whether reversal has occurred in the last three iterations.

2. Store the values of first mode spectral acceleration for
the previous iteration. This will be used as an upper or lower bound for the search once a convergence problem is detected.

3. When a convergence problem is detected, use a control flag to turn on the search routine in place of parts of the normal procedure.

4. Use the binary search method to search for the solution. During each trial, a constant first mode spectral acceleration is used until the program converges in the normal way.

5. Then check this acceleration against the acceleration from the spectrum, for the same period and the same level of damping. If the values do not agree then repeat step 4 with another trial value of constant acceleration. If the two values do agree within a certain limit of tolerance then the program has actually converged.

In designing the search routine for the program, it was also intended to minimize changes to the program as a whole. Therefore, only a control flag, 'LOCK', was added to the convergence checking routine in the main program and a subroutine 'STACHK' was incorporated into the main subroutine MOD3. The flowchart in Fig. 3.8 shows how STACHK is used in the main subroutine. It should be noted that STACHK only works for the first mode of the analysis. It will detect any reversal of the response period after the fifth iteration. If the reversal is larger than a predefined limit, the control flag will be set to 1. Thereafter,
subroutine STACHK will take over subroutine SPECTR and feed into the program a trial value of spectral acceleration. On the other hand, if no reversal in response period is detected, the program will go to subroutine SPECTR and collect the value of spectral acceleration in the normal way.

Fig. 3.9 is a detail flowchart of the search routine STACHK. Beginning from the fourth iteration, the response period and the spectral acceleration will be stored in memory. Starting from the sixth iteration, periods from the last two iterations will be compared using the following conditions

\[
T_{n-2} - T_{n-1} < -0.005 \quad \text{and} \quad T_{n-1} - T_n > 0.005 \quad (3.8a)
\]

\[
T_{n-2} - T_{n-1} > 0.005 \quad \text{and} \quad T_{n-1} - T_n < -0.005 \quad (3.8b)
\]

where \( T \) is the period in seconds

and \( n \) is the iteration number.

If either of the above conditions is met, then the binary search routine will be used. Upper and lower bounds will be set up using the values of spectral acceleration, at 2 percent damping, from the last and current iteration. The first trial value is the mean of the upper and lower bounds. This trial value will be used throughout the process until the program converges, which will be indicated by the value of IFLAG being set equal to one in the main program. However, this does not necessarily mean that the solution has been found, because the spectral acceleration, at the
current level of damping, may not be equal to that of the spectrum, at the same level of damping. Therefore, the following criterion is used to check the condition.

\[
\text{SADIF} = \left| \frac{S_{\text{cal}} - S_{\text{sp}}}{S_{\text{sp}}} \right| < 0.015 \quad (3.9)
\]

where \( S_{\text{cal}} \) is the trial value of spectral acceleration at the current level of damping \( S_{\text{sp}} \) is the spectral acceleration determined from the spectrum at the same level of damping.

If this criterion is not met, then the upper or the lower bound has to be adjusted according to these rules:

\[
S_{\text{n}}^u = S_{\text{n-1}} \quad \text{for} \quad S_{\text{cal}} > S_{\text{sp}} \quad (3.10a)
\]
\[
S_{\text{n}}^l = S_{\text{n-1}} \quad \text{for} \quad S_{\text{cal}} < S_{\text{sp}} \quad (3.10b)
\]

where \( S_{\text{n-1}} \) is the trial value of spectral acceleration, at 2% damping, from the last iteration \( S_{\text{n}}^u \) is the new upper bound value, at 2% damping and \( S_{\text{n}}^l \) is the new lower bound value, also at 2% damping.

Then the next trial value will, again, be the mean of these new bounds. This will carry on until criterion (3.9) is satisfied. Once it is satisfied, the control flag will be set equal to 2 and IFLAG remains equal to 1. This will finalize the search procedure. However, one extra iteration, as in the normal process, must still be performed in order to get the output of the results.
As a final comment, it must noted that the search routine is not perfect in all cases. First, the last iteration may sometimes increase the difference between trial and actual spectral acceleration. Second, the routine may require more 'accuracy' in the converged results. This would mean tighter limits to criteria (3.4) and (3.5). In the program the original limits are reduced by half once the search routine is in effect. This will require more iterations than before, but fortunately the program converges more rapidly as the upper and lower bounds get closer to each other. Thirdly, if the search routine fails, other than the 'BREAK' key on the terminal, the maximum number of iterations defined in the input data will be the only means to stop the program; therefore caution is needed when choosing this number. Finally, the limits in criteria (3.8a and b) may not be suitable for very small structures, because these tend to have minute fluctuations in period when convergence fails. It is uncertain whether the limits in criteria (3.8a and b) should be reduced to suit these structures since large structures might be able to converge within these limits with a few more iterations. Due to the rarity of this convergence problem, studies have been limited to only two or three cases.
3.5 Testing The Solution Searching Routine

The graph in Fig. 3.10 is used to illustrate the progress of the search routine in finding the solution for the example mentioned before. Since the routine used was an earlier version of what was presented above, the process started from the fifth iteration rather than the sixth and the limits stated in criteria (3.4) and (3.5) were used.

In this particular case, the spectral acceleration of the fourth iteration is the upper bound and that of the fifth iteration is the lower bound. The first guess is the mean value of the lower and upper bounds. The solution for that level of constant acceleration is point number 1 in Fig. 3.10, which is similar to point number 3 in Fig. 3.6. So, clearly, the constant acceleration is larger than that of the spectrum, and according to Eqn. (3.10a), the last trial value is treated as the new upper bound. The search goes on until criterion (3.9) is satisfied. In this case, three trials for the search routine and a total of eight iterations were needed to complete the search. The final result is at point number three in Fig. 3.10 where the period is 0.5779 sec. and the spectral acceleration is 0.2922g, as compared to 0.579 sec. and 0.292g in Freeman's method. The calculated spectral acceleration is different from that of the spectrum by only 0.8 percent. The results of this case are good, since the fluctuation in period is rather small. It is believed that reasonable results should
be attainable in other cases as well.
4.0 SUBSTITUTE DAMPING AND SMEARED DAMPING

4.1 General

The determination of structural damping is one of the most difficult problems in dynamic analysis (Ref. 7). Its value depends on a number of factors, such as the structural type, degree of damage that the structure undergoes, type of building materials used, as well as the participation of non-structural elements. Tests related to this area of study are generally limited to small frames or components and as such do not necessarily represent accurately the damping of real structures. In practice, damping is usually estimated on the basis of experience, bearing in mind the above factors.

4.2 Substitute Damping Factor

In the substitute-structure method, the formula for substitute damping (Eqn. 2.5) suggested by P. Gulkan and M.A. Sozen was developed from the results of a series of tests of reinforced concrete frames. All the frames tested
were portal frames in which a lumped mass was attached to the beam such that the units would represent single-degree of freedom systems. The dimensions of these frames were approximately 3 feet by 5 feet. The derivation of the formula is based on the assumption that the hysteresis loop of the system can be approximated by a linear system with equivalent viscous damping. According to L.S. Jacobson, this assumption would only be acceptable when dealing with systems that are almost linear and have small damping values; for strongly non-linear systems with non-viscous friction, the use of an equivalent viscous damping would be inadequate. However, L.S. Jacobson has also pointed out that most investigators generally prefer to represent the damping properties of any vibrating system in this manner, regardless of whether the system is linear or not.

The above assumption was also adopted throughout this study mainly because of its simplicity. Nevertheless, an attempt was made to derive an analytical formula for substitute damping that does not rely on test results.

Fig. 4.1(a) shows the system being considered for the derivation of the formula. In Fig. 4.1(b), a diagram similar to Fig. 2.1(d) is shown. The slope of OB represents the apparent stiffness of the real structure, which is also the actual stiffness of the substitute structure. By linearizing the problem in this way, the shaded area that is bounded by points OAB is neglected. Assuming that this shaded area is the energy lost by a member at maximum displacement, it is possible to express the amount of energy lost by the
following equation,
\[
E = \frac{[s (\mu-1) + 1]}{2 (1 - \mu s)} (\mu-1) M \theta_y
\]
(4.1)

where \( E \) is the energy lost by the member
\( \theta_y \) is the rotation at yield
\( s \) is the strain hardening ratio
and \( \mu \) is the damage ratio.

For the case of linear viscous damping which has been assumed in this study, the plot of damping versus displacement appears as an ellipse, such as is shown in Fig. 4.2(c), as suggested by L.S. Jacobsen\textsuperscript{7}, and R.W. Clough and J. Penzien\textsuperscript{9}. At resonance, the work being done per cycle by this damping force can be calculated as
\[
W_c = \pi c \omega_n \Delta^2
\]
(4.2)

where \( W_c \) is the work done by the damping force
\( c \) is the viscous damping coefficient
\( \omega_n \) is the natural frequency
and \( \Delta \) is the displacement of the portal frame.

The absolute damping factor, \( c \), can also be expressed in terms of a substitute damping coefficient, \( \beta_i \), and the critical damping factor, \( C_{\text{crit}} \). Thus,
\[
c = C_{\text{crit}} \beta_i
\]
(4.3)

but
\[
C_{\text{crit}} = 2 m \omega_n
\]
(4.4)
and \[ m \omega_n^2 = \frac{12 \, EI}{\mu \, L^3} \] (4.5)

Therefore Eqn. (4.2) can be rewritten as

\[ W_e = \frac{24 \, EI}{\mu \, L^3} \pi \, \Delta^2 \beta_i \] (4.6)

Now equate the amount of energy lost from the system to the work done by the damping force. This implies that the shaded area in Fig. 4.1(b) is equal to one quarter of the area of the ellipse in Fig. 4.1(c). Thus

\[ E = W_e / 4 \] (4.7)

By substituting

\[ \theta_y = \frac{M_y \, L}{6 \, EI} \] (4.8)

and

\[ \Delta = \frac{M_y \, L^2 \, \mu}{6 \, EI} \] (4.8)

into both sides of the expression in (4.7), a formula for \( \beta_i \) is derived. The equation is

\[ \beta_i = 0.02 + \frac{1}{2 \pi} \left[ \frac{s \, (\mu - 1) + 1}{\mu \, (1 - \mu \, s)} \right] \] (4.10)

A factor of 0.02, which represents the viscous damping before yield, is added to the equation.

If the strain hardening ratio, \( s \), is zero, then the above equation becomes
\[ \beta_i = 0.02 + \frac{1}{2 \pi} \left[ 1 - \frac{1}{\mu} \right] \] (4.11)

which is very similar to Eqn. (2.5) that is suggested by P. Gulkan and M.A. Sozen\(^5\). A comparative plot of Eqn. (2.5) and Eqn. (4.11) is shown in Fig. 4.2. From the figure, it is clear that the damping values obtained from Eqn. (4.11) are higher than those obtained from Eqn. (2.5). The application of the new formula therefore results in a reduction of the magnitude of the spectral acceleration and thus of the response of structure.

The values for substitute damping at 2 percent strain hardening, as obtained from Eqn. (4.10), are also plotted on Fig. 4.2. The figure indicates that damping increases more rapidly with damage ratio when strain hardening is included.

### 4.3 Smeared Damping Factor

After the substitute damping factor is determined for each member, the smeared damping factor of the structure for each mode can be calculated using Eqns. (2.6) and (2.7). These equations are still valid even though the formula for substitute damping has been changed. However, a comment regarding the reliability of these equations seems in order.

In the derivation of Eqn. (2.7), it is assumed that each member contributes to the modal damping in proportion to its relative flexural strain energy. The reason for this
assumption is not stated in the paper written by A. Shibata and M.A. Sozen\textsuperscript{2}. Even if the flexural strain energy is the correct weighting factor for determining the smeared damping, it is still doubtful whether the magnitude of the weight itself is general enough for all types of structures. Also as mentioned by P. Gulkan and M.A. Sozen\textsuperscript{5}, it is not justifiable to specify values of $\beta_s$ to more than two decimal places; this implies that it is rather impractical to demand a high degree of accuracy from the smeared damping equations. It is quite possible that this is a source of error in the present method. Further research in this area might prove fruitful.
5.0 ALTERATION TO THE DETERMINATION OF DAMAGE RATIOS

5.1 Member Stiffness Modification Using A Single Damage Ratio

'Damage ratio' is the key factor in the idea of using an elastic modal analysis to deal with an inelastic problem. Because of the use of this damage ratio, the complication of forming inelastic stiffness matrices and load incrementing is avoided. This implies that the stiffness matrices or formulas suitable for the usual structural analysis would also be applicable to the present method. In the program, three standard member stiffness matrices are used (Fig. 5.1). They are the axial stiffness matrix (a), the bending stiffness matrix (b), and the rigid arm stiffness matrix (c). Since the use of these matrices requires that the members be linear elastic, all stiffnesses involving the term EI have to be divided by the damage ratio as shown in Fig. 5.1. The object of this is to reduce the stiffness of the member in order that the loss of stiffness due to plastic action can be taken into account.

In the previous program, the damage ratio is calculated and modified in subsequent iterations by using the bigger of the two end moments in the member. Then the sectional
rigidity of the member is modified by this damage ratio. In
doing this, it is implied that the whole member is damaged
by an amount equal to this bigger damage ratio at one end.
This argument is acceptable only if the end moments are
equal or very close to each other, which in fact is true for
most members in a typical frame, except the base columns and
the edge beams. However, it can be shown later that this
argument is only a rough approximation even if the end
moments are equal. For other types of structures, such as
coupled frame shear-wall structures, the variation in
bending moment along the structural wall could be quite
different to what is assumed above. In such cases, large
discrepancies would occur at the base of the structural wall
since the stiffness for members at the base is grossly mis-
represented by the above assumption.

It is believed that the removal of such a
simplification could improve the reliability of the results
obtained from the present method. This can be done by
applying one damage ratio to each end of the member. The
trade-off in doing so would be added complexity in the
program and thus higher cost in running the program.
However, the improvements brought about by the alteration
are rather significant, as can be shown by the results in
Chapter 6.
5.2 Member Stiffness Modification Using Damage Ratio At Two Ends

In order to remove the drawback of the simplified procedure, a new model is introduced so that the damage ratio of both ends can be taken into consideration. The new model will involve hinges at each end of the member. A typical member is shown in Fig. 5.2.

The sectional rigidity along the length of the hinge will be a function of the damage ratio at both ends. Assuming that the sectional rigidity of the hinge at joint A and that at joint B is

\[ EI_A = A \cdot EI \]  
\[ EI_B = B \cdot EI \]  

respectively, where EI is the initial sectional rigidity of the member; A and B are factors that are used to reflect the damage of the joint by reducing the initial sectional stiffness as damage occurs. Factors A and B will always be less than one when damage ratios are greater than one.

To derive a relationship between the two factors and the damage ratio of the member ends, the following assumptions become necessary:

1. In accordance with basic structural theory, when a unit rotation is applied to the pinned end of a member while the other end is held fixed, the moment thus created at the pinned end would be \( 4EI/L \). If the damage ratio at
that end is $\mu$, then the corresponding moment or stiffness will be $4EI/(\mu L)$.

2. When damage occurs to a joint, a plastic hinge with hinge length equal to one twentieth of the member length will develop. (It is thought that this will be approximately equal to the member depth.)

3. The sectional stiffness along the length of the hinge is constant.

From the first assumption, it is clear that, for joint A, the moment due to a unit rotation at the same joint would be $4EI/(\mu_A L)$, and for joint B, it would be $4EI/(\mu_B L)$, where $\mu_A$ and $\mu_B$ are the damage ratios at joint A and joint B, respectively. Thus,

$$ k_{33} = \frac{4 EI}{\mu_A L} \quad (5.3) $$

$$ k_{66} = \frac{4 EI}{\mu_B L} \quad (5.4) $$

In many non-linear dynamic analysis programs, a plastic hinge with zero hinge length is adopted. The use of such an assumption would generally simplify the procedure in establishing non-linear stiffness matrices. However, this assumption is unrealistic due to the fact that plastic hinges are known to have a certain length. In normal cases, the plastic hinge would develop from the end of the member to a distance equal to the depth of the beam. So for a typical beam with depth to length ratio of 0.05, the hinge...
length would be approximately equal to the depth of the member.

For an ordinary 6 by 6 stiffness matrix, there should be 36 different damage ratios. However, all these stiffnesses can be expressed in terms of \( k_{33} \), \( k_{66} \), and \( k_{63} \) (Ref. 11), therefore only the damage ratio for \( k_{63} \) has to be found. This is done by using the conjugate beam method.

A diagram showing the real and conjugate beam is given in Fig. 5.3 to help explain the method. Fig. 5.3(a) is the real beam with a unit rotation at joint A. The moment diagram of the beam is shown in Fig. 5.3(b). The conjugate beam corresponding to the real beam will be as shown in Fig. 5.3(c). To produce the elastic load of the conjugate beam in Fig. 5.3(d) and also the equations of equilibrium, the following rules are applied according to the conjugate beam method:

1. The curvature of the real beam is equal to the distributed load of the conjugate beam.
2. The slope of the real beam is equal to the shear in the conjugate beam.
3. The deflection of the real beam is equal to the moment in the conjugate beam.

To determine the distributed load in the conjugate beam, superposition of the different component parts is applied. These components are shown in Fig. 5.4. The first two elastic load components are caused by the moments at the two ends, neglecting the hinges. The next two components account for the effect of the hinge at joint A and the last two
correspond to hinge at joint B. The sum of all these produces the total distributed elastic load at the bottom of the figure.

Two equations of equilibrium can be obtained by taking moments about joint A and joint B. According to rule 3, the sum of moments at the two ends should be zero, since the end deflections of the real beam are zero. After reorganizing the equations, the following equations, written in terms of factors A and B, are obtained

\[
M_A = k_{33} = \frac{EI \left[ 0.2858AB + 0.000417B + 0.04754A \right]}{L} \quad \text{(5.5)}
\]

\[
M_B = k_{63} = \frac{EI \left[ 0.1643AB + 0.01208(A+B) \right]}{L} \quad \text{(5.6)}
\]

where \( f(A,B) = \)

\[
0.00226 + 0.0547AB + 0.0132(A+B) + 5(10)^{-7} \frac{A^2+B^2}{AB} \quad \text{(5.7)}
\]

From the configuration shown in Fig. 5.3(a), it is clear that \( M_A \) is the same as \( k_{33} \), therefore from Eqn. (5.3) and (5.5), the following equation is derived

\[
\mu_A = \frac{4 f(A,B)}{\left[ 0.2858AB + 0.000417B + 0.04754A \right]} \quad \text{(5.8)}
\]

A similar equation can also be derived for \( \mu_B \) by interchanging joint A and joint B in Fig. 5.3(a). Thus,
\[
\mu_B = \frac{4 f(A,B)}{[0.2858AB + 0.0000417A + 0.04754B]} \quad (5.9)
\]

By assuming that

\[
k_{63} = \frac{2 EI}{\mu_e L} \quad (5.10)
\]

Eqn. (5.6) gives

\[
\mu_e = \frac{2 f(A,B)}{[0.1643AB + 0.01208 (A+B)]} \quad (5.11)
\]

where \(\mu_e\) is the unknown damage ratio and will be referred to as the 'effective' damage ratio from hereon.

The next step is to derive an equation for \(\mu_e\) in terms of \(\mu_A\) and \(\mu_B\). Although there are enough equations for the derivation, it is unlikely that this can be done directly from the above equations. Therefore, an iterative method was adopted to determine \(A\) and \(B\) from Eqn. (5.8) and (5.9); then, the values for \(\mu_e\) can be obtained from Eqn. (5.11). Since \(\mu_A\) and \(\mu_B\) would be known quantities during the analysis, a range of values for \(\mu_A\) and \(\mu_B\) can be assumed here. The range of damage ratios used is from one to six with an increment of one, for both \(\mu_A\) and \(\mu_B\). The results are tabulated in Table 5.1. From the first six lines of the table, it is interesting to find that joint A becomes more rigid as the damage ratio in the other joint increases. This has contributed some error to the values in \(k_{33}\), which should be equal to 4. However, the error is small and
therefore can be accepted. Other values for \( k_{33} \) are in close agreement with Eqn. (5.3).

Fig. 5.5 is a plot of \( \mu_e \) versus \( \mu_A \) and \( \mu_B \). It shows that \( \mu_e \) is an increasing function of both \( \mu_A \) and \( \mu_B \). A comparison with the simplified method used previously is also made in Fig. 5.5. The dotted lines at the bottom of the graph with \( \mu_A \) ranging from one to six represent the values obtained from the simplified method. The difference between the previous simplification and the present model is obvious, and the difference becomes more pronounced as both \( \mu_A \) and \( \mu_B \) increase. Therefore, the reliability of results generated by the previous method is rather limited as damage ratios of the structure become larger.

The final step of deriving the equation is to fit the lines in Fig. 5.5 by an appropriate formula. Fortunately, the relationship among \( \mu_A \), \( \mu_B \) and \( \mu_e \) is linear, hence, it can easily be done by using the least-square method. First, the six lines in the figure are fitted with straight lines which will produce six line equations relating \( \mu_B \) to \( \mu_e \) for different values of \( \mu_A \). Then by fitting straight lines through the six constants and slopes, separately, an equation for the constants and an equation for the slopes, both written in terms of \( \mu_A \), is obtained. Putting these equations back into the expression for \( \mu_B \) and \( \mu_e \) will yield an equation for \( \mu_e \) in terms of \( \mu_A \) and \( \mu_B \). However, this equation must be adjusted slightly, due to errors in line fitting, so that \( \mu_e \) would be equal to one when both \( \mu_A \) and \( \mu_B \) are unity. The final expression is
\[ \mu_e = 0.095 + 0.2(\mu_A + \mu_B) + 0.505\mu_A \mu_B \quad (5.12) \]

This equation is symmetrical in terms of \( \mu_A \) and \( \mu_B \), which has eliminated any possibility of having directional effects.

The final step is to put these three damage ratios into the stiffness matrix. This can be done, first, by substituting Eqn. (5.3), (5.4), and (5.10) into the matrix (Ref. 11) shown in Fig. 5.6 where all flexural terms are expressed in terms of \( k_{33} \), \( k_{66} \) and \( k_{63} \). However, the matrix is formed with respect to the member local axes only. So the next step would be to transform the matrix so that it can be used in any arbitrary set of reference axes. The transformed stiffness matrix can be determined by substituting the results obtained from the last step into the matrix in Fig. 5.7. The final matrix is a combination of the bending and axial stiffnesses. By comparing it with the matrix in Fig. 5.1(a) and (b), damage ratios for all the terms are obtained. There are a total of six damage ratios, but, of course, four of them will be functions of \( \mu_A \) and \( \mu_B \).

For ease of programming, the reciprocal of these damage ratios is used as a multiplication factor for each stiffness term. The following are the two basic factors,

\[ \mu_1 = 1/\mu_A \quad (5.13) \]
\[ \mu_2 = 1/\mu_B \quad (5.14) \]

which only apply to 4EI/L terms. The factor for 2EI/L terms is
\[ \mu_3 = 1/\mu_e \]  

then, the other three factors are

\[ \mu_4 = \frac{1}{3} \left[ \frac{2}{\mu_A} + \frac{1}{\mu_e} + \frac{1}{\mu_B} \right] \]  

\[ \mu_5 = \frac{1}{3} \left[ \frac{2}{\mu_e} + \frac{1}{\mu_A} + \frac{1}{\mu_B} \right] \]  

\[ \mu_6 = \frac{1}{3} \left[ \frac{1}{\mu_A} + \frac{1}{\mu_e} + \frac{1}{\mu_B} \right] \]  

The stiffness terms where these factors apply are shown in the bending stiffness matrix given in Fig. 5.8. The factor, \( h \), which accounts for shear deflection, is not modified because of the difficulties in obtaining a proper modification factor for it. Therefore, it is assumed that shear deflection of the structure is not affected by flexural damage.

Finally, terms in the rigid arm stiffness matrix are modified in the same manner as appeared in the bending stiffness matrix.

5.3 Alteration In The Calculation Of Strain Energy And Substitute Damping

5.3.1 Strain Energy

In the present method, the member strain energy is used as a weight factor so that the smeared damping of the
structure can be determined from the substitute damping factor of the members. Eqn. (2.6) was used to calculate the strain energy in the previous method (Ref. 2, 3), where

$$\Pi = \frac{L}{6(EI_a/\mu)} \left[ M_A^2 + M_B^2 - M_A M_B \right]$$ \hspace{1cm} (2.6)

where $EI_a$ is the section rigidity for the element in the actual structure.

This equation is derived from the strain energy, which is defined as

$$\Pi = 0.5 \begin{bmatrix} M \end{bmatrix} \begin{bmatrix} f \end{bmatrix} \begin{bmatrix} M \end{bmatrix}$$ \hspace{1cm} (5.19)

where

$$[M] = \begin{bmatrix} M_A \\ M_B \end{bmatrix}$$

and

$$[f] = \frac{L}{EI} \begin{bmatrix} 1/3 & -1/6 \\ -1/6 & 1/3 \end{bmatrix}$$

which is the flexibility matrix.

By analogy, a similar formula can be derived for the new model. The first step is to invert part of the stiffness matrix that was found in the last section to yield the flexibility matrix. By inverting

$$[k] = \frac{EI}{L} \begin{bmatrix} 4/\mu_A & 2/\mu_e \\ 2/\mu_e & 4/\mu_B \end{bmatrix}$$ \hspace{1cm} (5.20)

this will give

$$[f] = \frac{L}{\left[ \frac{16}{\mu_A \mu_B} - \frac{4}{\mu_e^2} \right] EI} \begin{bmatrix} 4/\mu_B & -2/\mu_e \\ -2/\mu_e & 4/\mu_A \end{bmatrix}$$ \hspace{1cm} (5.21)
Then, putting \([f]\) into Eqn. (5.19), a new expression for strain energy is derived. This gives

\[
\Pi = \frac{2L}{[\frac{16}{\mu_A \mu_B} - \frac{4}{\mu_B^2}]} \frac{M_A^2}{EI} + \frac{M_B^2}{\mu_A} - \frac{M_A M_B}{\mu_e} \quad (5.22)
\]

This equation is much more complex than Eqn. (2.6) because of the involvement of another damage ratio. The above equation, if both \(\mu_A\) and \(\mu_B\) are equal to one, will degenerate to Eqn. (2.6).

5.3.2 Substitute Damping

The equation derived in Chapter 4 is still valid in the new model. The only change to be made is to replace the single damage ratio by the average of the damage ratios at the two hinges. Therefore,

\[
\beta_i = 0.02 + \frac{1}{2\pi} \left[ \frac{s (\mu'-1) + 1}{\mu'} (\mu'-1) \right] \quad (5.23)
\]

where \(\mu'\) is the average of \(\mu_A\) and \(\mu_B\).

5.4 Computer Program

To incorporate all these modification into the computer program would require several changes. First of all, the subroutine DAMOD has to be altered so that it can modify two
damage ratios for each member. Both damage ratios have to be checked by criterion (3.5a and b). However, only the bigger moment of the two ends is checked by criteria (3.4) in order to avoid double counting of the number of members above capacity.

Major changes have to be made to subroutine BUILD and FORCE, where the stiffness matrix is assembled and forces are calculated from displacements. Calculations for factors in Eqns. (5.13) through (5.18) are performed in a new subroutine DAMCAL.

Fig. 5.9 and 5.10 are the final revised flowcharts for the main program and subroutine MOD3, respectively, showing all the major changes made to the program in this study.
6.0 THE PRESENT METHOD VS. INELASTIC TIME-STEP ANALYSIS

6.1 General

There are several ways of testing the present method for its ability to predict the inelastic response of structures. The best way to do this is, of course, to feed into the program the model of a real structure which has survived a severe earthquake and for which both the ground excitation and the response of the structure have been recorded. Then the predicted results can be compared with the actual measurements to determine the accuracy of the program.

In fact, records of this type are available from real life events or can be obtained from small scale experiments. Although the former provides the best means of comparison, the response records are usually limited to only a few points in the structure. Therefore, the reliability of the program may not be justified by this kind of comparison while, at the same time, a considerable amount of effort has to be spent on modelling the real structure. Small scale tests provide more information under controlled laboratory conditions, but the result is probably inadequate when applied to large scale structures. Besides, it is rather
impractical, especially when only the peak response is required, to conduct this kind of experiment. A time-step analysis can produce sufficiently accurate results for the purpose of this study, and such a program was chosen to calibrate the present method.

6.2 Comparison Of The Two Methods

6.2.1 Advantages And Disadvantages Of The Two Methods

Time-step analysis is capable of producing a complete response time history for a given structure and a given earthquake record. It is the completeness and the accuracy of the analysis that give it the advantages over all the other methods. Nevertheless, the cost of execution imposes a major setback for this kind of analysis.

The execution time (CPU time) of a time-step analysis depends largely on the length of the digitized earthquake record, the size of each integration time-step and the number of degrees of freedom to be analysed. The actual running time would, of course, be much longer than the execution time. For some test structures in this study that were analysed by the time-step analysis program, DRAIN-2D, using an eight second record and 0.0125 second time-step, took more than an hour to complete in a high priority job, yet EDAM2, the program for the proposed method, can complete
the analysis for the same structure in less than ten minutes. This has made time-step analysis inferior especially when dealing with parametric studies and system optimization.

As was mentioned before, the choice of representative earthquake records added another disadvantage to this type of analysis. In preparing the input data, a modal analysis is required to determine the fundamental period of the structure. This period may be used to calculate the damping factor for the input structure. Hence, data preparation for the analysis is also rather time consuming.

Although time-step analysis can provide a complete response time history of a structure, the interpretation of the results is left open to each individual who uses the program. In practice, the resulting sets of forces and displacements are usually converted to ductility ratios. However, different definitions of ductility ratio lead to different interpretation of the results; this is also a disadvantage of the analysis.

In contrast, the adoption of the damage ratio in the present method has removed the ambiguity of using ductility ratios. Also, due to the advantages in execution speed and execution cost of the present method, it is more suitable for parametric studies.

However, there are also disadvantages in the present method mainly related to modal analysis. Firstly, there is no exact way of combining the response of each individual mode to yield the overall response. Secondly, some objection
has been raised against modal analysis owing to the fact that the duration of the earthquake is not being taken into account.

6.2.2 Comparison Of The Two Programs

DRAIN-2D is a general purpose time-step analysis program developed at the University of California at Berkeley. The program has been well tested and documented. It has been used widely to perform dynamic analysis of a large variety of structures and is known to have produced good results. The following is a comparison between DRAIN-2D and EDAM2.

In general, both the code of the program and the required input for DRAIN-2D are more complex than that of EDAM2. However, DRAIN-2D offers more flexibility over the available member types and the ways that member properties are specified. In DRAIN-2D, there are six types of elements to choose from which include a truss element, two types of beam column elements, an infill panel element, semi-rigid connection element and beam element with degrading stiffness. The last element type was used in this study. The characteristics of this element are represented by modelling the hysteresis loop with stiffness degrading through each cycle of response. In using this type of element, the user has a choice of adopting the original Takeda model or the modified Takeda model. The original Takeda model was chosen in this study, for simplicity.

At this time, EDAM2 can only support two basic types of
elements, a truss element and a frame element. On the other hand, DRAIN-2D also allows: members to have different moment capacity and stiffness at the two ends, the inclusion of P-Δ effect, and input of static loads that apply to the structure. In order that the two programs can be more comparable, these functions are by-passed in performing the time-step analyses. However, some of these differences are considered as potential areas for improving EDAM2 and can be incorporated into the program later on, if necessary.

6.2.3 Limitations In The Comparison Of Results

Since the bases of the two methods are different, difficulties in obtaining a fair comparison between the results are evident. The following are some major differences between the two programs that are worth noting.

Firstly, the ways of handling damping are different between the two methods. Even so, no attempt was made to match the damping obtained from two different models, since the accuracy of the Takeda Model itself is limited.

Secondly, through a careful study of the documentation for the element with degrading stiffness, a major difference in the definition of strain hardening ratio was unveiled. It was found that the strain hardening ratio defined in EDAM2 is equivalent to 3/4 of that applying to DRAIN-2D. At the same time, an expression used to calculate damage ratio from the results of DRAIN-2D is also developed. This expression is
where the yield moment, maximum moment and strain hardening ratio are the only required quantities. Therefore, only the envelope of the maximum displacements and forces from the output is needed, whereas the lengthy response time history can be neglected in calculating damage ratios.

Finally, a zero plastic hinge length is assumed in DRAIN-2D, while non-zero hinge length is adopted in EDAM2. This represents a major difference in the modelling of an actual structure.

6.3 Choice Of Earthquake Record

Nowadays, a large number of digitized earthquake records are available for use in dynamic analysis. El Centro, 1940, is one of the popular records that is used widely in structural design and analysis. However, the highly irregular shape of the spectrum for this and many other records imposes great difficulty in obtaining a smooth spectrum while retaining the characteristics of the real spectrum. In order to obtain a better comparison between the two methods, an earthquake record that will produce a smooth spectrum is essential. One such record was found in a research report concerning simulated earthquake motions\textsuperscript{13}. A total of eight digitized records were generated using the

\[
\mu = \frac{4}{3} \left( \frac{M_n - M_y}{s M_n} \right) + \frac{M_y}{M_n} \tag{6.1}
\]
method presented in this paper, where C2 is one of them. The duration of the record is 12 seconds and its spectrum, shown in Fig. 6.1, is rather smooth especially in the region of longer period.

As can be seen from Fig. 6.1, the spectral acceleration varies significantly from the period of 0.2 to 0.5 sec., which is the approximately constant branch of the spectrum. Smoothening that region of the spectrum is important to the accuracy of the high modes. Since it is not possible to have pinpoint accuracy in the calculation of modal response periods from EDAM2, some averaging of the spectral acceleration must be imposed in that region, so that a slight error in response period would not create a large difference in the value of spectral acceleration.

In fitting the earthquake spectrum, the curve corresponding to 2 percent damping was used. These lines are then input into the computer program. It is found that the scaling equation (Eqn. (2.8)) is not always accurate; the effect of scaling a spectrum from 2 percent damping to 10 percent damping is shown in Fig. 6.1 and Fig. 6.2.

On the other hand, the length of the earthquake record to be input into time-step analysis must also be chosen appropriately. The time when the earthquake acceleration is high does not necessarily coincide with maximum response of the structure because of the difference in period between the earthquake motion and the structure. Therefore the earthquake record input to the time-step program must be long enough to cover the time where the maximum response
occurs. Or else, the response spectrum must be generated from the same earthquake record with the same duration that is used in the time-step analysis.

To illustrate the usefulness of the program with other earthquake records, a test was conducted using the 1971 San Fernando earthquake. The earthquake record obtained in the ground floor of 8244 Orion Ave., Van Nuys, Holiday Inn was used. Its spectrum is shown in Fig. 6.2. It is obvious that the spectral acceleration varies significantly with the response period, which is typical in many real earthquake records. In spite of that, reasonable results were obtained from the proposed method; the results will be shown in Section 6.4.5.

6.4 Test Results And Discussions

6.4.1 General

Basically, three different structures were designed so that the results from the present method and from time-step analysis could be compared. The first test structure is an 8-bay, 7-storey frame; the second test structure is a 5-bay, 7-storey shear-wall frame and the last test structure is an irregular frame resembling the first test frame but with the left portion of the upper 3 storeys cut off to give a setback.

Tests on these structures were divided into three
phases. In the first phase (Section 6.4.2 to 6.4.4), all three structures were analysed once by both programs using the C.I.T. simulated earthquake, C2, record. In the second phase (Section 6.4.5 to 6.4.7), some parameter changes were made so that the effects of these parameters and the reliability of the present method could be investigated. In the final phase (Section 6.4.8), a comparison between results from the original program (EDAM), the new program (EDAM2) and DRAIN-2D was made. This was done simply by running the first phase tests again on EDAM. This comparison shows the effect of applying the new set of damping formulas in Chapter 4 and also the effect of using different damage ratio at two ends of a member.

Since smooth convergence was attained in all of the six tests within the first and second phase, the solution searching routine mentioned in Chapter 3 was not tested in this study.

6.4.2 Test Frame No. 1

Test frame no. 1 is an 8-bay, 7-storey frame resembling the Holiday Inn described in Ref. 14. The dimensions of the test frame are given in Fig. 6.3. The ground storey is 13.5' in height, second to sixth storey are 8.708' high, and the top storey is 8.67' high. Columns are spaced equally at 18.75'. The properties of this test frame, as well as the other test structures, are tabulated in table 6.1. For test frame no. 1, the fundamental period is 0.78 sec. Floor weight decreases from the first to the top floor. The
exterior columns are smaller than the interior columns. Sectional properties for columns and beams were obtained by summing the properties of four individual frames together in the transverse direction; therefore, the numbers will appear to be larger than any ordinary single frame. Since it was assumed that the floor slab can act as a diaphragm, an arbitrary large number was assigned to the beam areas.

First, an elastic modal analysis was performed on the test frame to determine the earthquake load for each member. In doing the analysis, the NBC spectrum at 5 percent damping and a ground acceleration of 0.25g was used. A ductility of 6 was applied to all beams and a ductility of 2 to all columns in the initial design; this was to ensure that beams yield before columns. Then the moment capacity of each member, shown in Fig. 6.3, was obtained according to design code equations.

In this test, the C.I.T. simulated earthquake, C2, motion was applied at a maximum acceleration of 0.3g. A strain hardening ratio of 1.5 percent was also used in EDAM2; this was equivalent to 2 percent strain hardening in DRAIN-2D. The results of the test were presented in Fig. 6.4, Fig. 6.5 and summarized in Table 6.2. Fig. 6.4 shows that damage only occurred in beams as predicted. The distribution of damage ratios was highly non-uniform. From Fig. 6.5, it is clear that the maximum damage of the frame appeared on the first floor and damage decreases as floor level increases. Results predicted by EDAM2 were rather conservative at the upper levels, especially on the fifth
floor, but were unsafe at the lower levels. On the right hand side of the figure is a plot of horizontal displacement versus floor level. The maximum displacement predicted was 3.27" from EDAM2 and 3.94" from DRAIN-2D. Although the discrepancy in horizontal displacement was quite high, the present method was able to predict the deflected shape of the test frame.

Table 6.2 shows a summary of the above results. The inelastic response period was 1.14 sec. The maximum error in base shear was 7.7 percent which was the largest among all the tests conducted in this study. The maximum error in horizontal displacement was 36.1 percent and the maximum absolute error in damage ratio was 1.67, occurring in the first floor. The maximum percentage error in damage ratio, however, was on the fifth floor. Even though the errors in damage ratio seem large, the present method was capable of picking out the trouble spots in the test frame, which is regarded as one of the goals in developing the method.

6.4.3 Test Frame No. 2

Test frame no. 2 is a shear-wall frame. Its dimensions and the yield moment for each member are shown in Fig. 6.6. The shear wall was idealized as line members on the wall centre-line, connected to the beams by rigid arms. These columns had the capacity and sectional properties of the actual shear wall. The attached frame had the same dimensions as test frame no. 1, except 5 bays were used instead of 8. The structural properties of the test frame
are given in Table 6.1. The fundamental period for the test frame was 0.51 sec. The properties shown in the table correspond to one single frame. The ductility assigned to columns and beams was, again, 2 and 6, respectively. The C.I.T. Simulated earthquake, C2, motion was used at a maximum acceleration of 0.3g. Only a very small amount of strain hardening was applied to simulate an elastoplastic condition so that the effect of strain hardening can be isolated. The reason for applying non-zero strain hardening was because, for the degrading stiffness element, DRAIN-2D will produce an infinite displacement response as strain hardening goes to zero.

The results obtained from EDAM2 and DRAIN-2D are shown in Fig. 6.7 and 6.8. Again, all the beams yielded whereas all the columns remained elastic, except at the base of the shear wall, which was the plastic hinge area of the wall. It was also the area where the maximum error in damage ratio was found. Fig. 6.8 shows that the damage ratios predicted by EDAM2 were generally on the low side, however, the trend in distribution of the damage was predicted successfully. On the other hand, the horizontal displacements obtained from both methods were in close agreement as can be seen from Fig. 6.7. The maximum horizontal displacement predicted by EDAM2 and DRAIN-2D was 2.17" and 2.48", respectively.

A summary of the test results is given in Table 6.2. It can be seen that the change in response period was smaller in magnitude than the last test frame even when the strain hardening ratio is lower. This indicated the stiffening
effect of using a shear wall instead of a bare frame. The errors shown in Table 6.2 were low for this test; the largest error in horizontal displacement and damage ratio was generated at the base of the shear wall.

6.4.4 Test Frame No. 3

Test frame no. 3 is a 7-storey frame with the upper left portion cut off. This test frame is used to examine the necessity for restricting abrupt changes in geometry in the height of the structure. The dimensions and yield moments are shown in Fig. 6.9 and the structural properties are shown in Table 6.1. The structural properties of this frame were exactly the same as test frame no. 1. However, the fundamental period of this test frame was lower than the first test frame due to the part being cut off.

The resulting damage ratios of this test are shown in Fig. 6.10 and Fig. 6.11(a). In general, results obtained from EDAM2 were conservative especially at the base of the structure. DRAIN-2D predicts that all columns should remain elastic, but EDAM2 predicts that the top level columns would slightly exceed their elastic limit. Despite the discrepancy between the results, the trend as the damage ratio varies from floor to floor was faithfully reproduced by EDAM2, as can be seen from Fig. 6.11(a). Fig. 6.11(b) shows the horizontal displacements of the test frame. The tip displacement was 3.25" and 2.76" as predicted by EDAM2 and DRAIN-2D, respectively.

From Table 6.2, the maximum errors were about normal,
at least not as high as the first test frame. These errors occurred mainly in the values at the base of the structure.

As shown by the results of this test frame, as well as the last two frames, it is apparent that the weakness of EDAM2 is the inability to predict the response at the base of a structure accurately. But, on the other hand, EDAM2 is very successful in predicting the pattern of damage that occurs in a structure and it is also able to predict base shear accurately within 10 percent error. This ends the first phase of calibrating EDAM2 against DRAIN-2D with the three basic test frames.

6.4.5 Test Frame No. 4

Test frame no. 4 is actually the same as test frame no. 1. The difference in this test was in the earthquake record, where the C.I.T. simulated earthquake record was replaced by the San Fernando earthquake. A maximum ground acceleration of 0.2g was applied. This is to test the ability of EDAM2 to produce consistent results with different earthquake spectra. Since the San Fernando earthquake record produces a very jagged response spectrum, a number of different smooth spectra can be chosen. Therefore, it can also test the sensitivity of the method to different smooth spectra representing the same record.

Fig. 6.12 shows the damage ratios and horizontal displacements, and Fig. 6.13 shows a plot of the beam damage ratio versus floor level. The agreement of the results was very good, although results obtained from EDAM2 were
slightly conservative. Values of maximum error given in Table 6.2 also reflected significant improvement over the first test. Therefore, consistent results from the present method are attainable if the input response spectrum is appropriate.

6.4.6 Test Frame No. 5

Test frame no. 5 is the shear-wall frame that was mentioned in Section 3.3. A ductility of 4 was assigned to both beams and columns. This will affect the yield moments of the test frame whereas other properties, including the elastic response period, will remain the same as test frame no. 2. The yield moment of each member is shown in Fig. 6.15. The C.I.T. simulated earthquake motion was used again in this test. However, the maximum ground acceleration applied was 0.2g since the test frame was not able to sustain 0.3g without a large amount of damage.

The test results in fig. 6.15 show that most of the columns had yielded. A large portion of the force at the base was absorbed by the shear wall and thus yielding was avoided in some base columns. Beams connected to the shear wall also yielded. As seen from Fig. 6.15 and Fig. 6.16, the agreement between the results was outstanding. However, past experience has shown that the present method tends to yield better results as long as the damage of the input structure stays relatively low; therefore, it was not surprising that these results were obtained.

In this test, the best agreement was in the prediction
of base shear, where the match was almost exact. The worst point was again at the base of the shear wall. Although the difference in damage ratio at the base of the shear wall was 0.8, the magnitude of the damage ratio was only about 3, compared to about 6 in test frame no. 2; hence, in terms of relative error, this discrepancy was actually comparable to that of test frame no. 2.

In concluding this particular test, it must be noted that this test frame is an exceptional case, in that EDAM2 converged successfully. Similar changes made to the other two frames led to the development of a soft storey, that caused a convergence problem.

6.4.7 Test Frame No. 6

Test frame no. 6 is the same as test frame no. 2. All the details were the same except that the strain hardening ratio for the test frame was changed from 0.075 percent to 1.5 percent in EDAM2 (from 0.1 to 2.0 percent in DRAIN-2D) in order to observe the effect of incorporating strain hardening in the method.

Figs. 6.17 and 6.18 show the results of this test. When comparing these two figures with Fig. 6.7 and 6.8, several differences are noticed. Firstly, the damage ratios and the horizontal displacements are noticeably smaller while the base shear, on the other hand, is larger. Secondly, the agreement between the results is not as close as in the second test, even though EDAM2 was still able to predict the distribution of damage ratios. Finally, the discrepancy
between beam damage ratios in the first floor was more pronounced.

Table 6.2 indicates that the inelastic response period was smaller than test frame no. 2. It also shows a general increase in maximum error. Although the comparison was not as favourable as that without strain hardening, the results should be acceptable.

6.4.8 A Comparison Of Results Between EDAM, EDAM2 And DRAIN-2D

The final phase of testing was to analyse the three basic test frames again using the original EDAM program and to compare the results with the new version, EDAM2, and DRAIN-2D. In order to obtain a fair comparison, the effect of strain hardening was incorporated into EDAM.

The results in beam damage ratios of test frame no. 1 are given in Fig. 6.19. There was a general shift in damage ratio, where relatively higher damage ratios were predicted by EDAM. The results from EDAM compare more favorably to DRAIN-2D than EDAM2 on the first two floors, but it was too conservative in other areas. When the damage ratios in columns (not shown) were compared, it was observed that the values from EDAM were approximately 4 percent higher than those from EDAM2 at the base of the test frame and about the same in other areas.

The next test was to re-analyse test frame no. 2 by EDAM. Fig. 6.20 shows a plot of beam damage ratios obtained from the three programs. Again, results from EDAM were a bit
better than those of EDAM2 at the base, but not as good as EDAM2 anywhere else on the frame. A comparison of column damage ratios revealed that values from EDAM were significantly lower in the ground floors. It was particularly important to note the difference in damage ratios at the base of the shear wall. EDAM predicts a damage ratio of 3.37 which is very low compare to 5.67 from EDAM2 and 7.17 from DRAIN-2D. This discrepancy was caused by assuming equal moment at the two ends of the base column.

Finally, test frame no. 3 was analysed by EDAM. Results were shown in Fig. 6.21. There was, again, a shift in beam damage ratio, as occurred in the last two tests, but this time the damage ratios were too conservative, especially at the base of the test frame. Column damage ratios were rather similar to those obtained from EDAM2.

Table 6.2 shows the summary of test results from EDAM for test frame no. 1', 2' and 3'. It is shown in the table that, in using the new version, generally, the resulting inelastic response period is shorter, whereas the smeared damping is higher, than that obtained from the previous version. The former is likely caused by the introduction of two damage ratios to a member, which changes the stiffness matrix that determines the response period, whereas the latter is the effect of using the new set of damping equations (see Fig. 4.2). Although the errors in base shear are small for EDAM, the errors in horizontal displacements and damage ratio are high. Also from these comparisons, EDAM2 seems to produce more consistent results than EDAM.
6.5 Cost Of Execution

The computer used for all of the above analyses was an Amdahl V/6 II which is comparable with an IBM 360/370. The cost of execution discussed in the following paragraphs refers to the same computer. In order to isolate other effects, such as different rates charged at different times of the day etc., the CPU time is used for the following discussion.

The execution cost of EDAM2 depends on a number of factors, such as the number of modes, number of degrees of freedom, half bandwidth of stiffness matrix and the number of iterations required for the program to converge. For DRAIN-2D, the factors are quite similar, but instead of number of modes and number of iterations, it depends on the number of time steps required to go through the complete duration of the input earthquake record. In other words, a longer record will cost more, if the step size is being held constant. Also, as more members go into the inelastic range, the cost in running DRAIN-2D increases, because of the more complex routine used to deal with inelastic action.

Most of the above factors are summarized in Table 6.3. In using the C.I.T. simulated earthquake record, only the first 5 seconds out of the 12-second record is required, since the maximum response usually occurs within the second and the fourth second of the record. This has given DRAIN-2D an advantage in the cost comparison. However, when the San
Fernando earthquake record is used, the first 15 seconds of the record are required, which in fact is very common among popular earthquake records, such as El Centro or Taft. The CPU time required for EDAM2 to complete the fourth test was 10.58 sec. and it took 159.05 sec. for DRAIN-2D to complete the analysis on the same structure. In another words, using the present method instead of a time-step analysis would save up to approximately 15 times in computing cost.

By comparing tests on frames no. 3 and 6, in Table 6.3, it is seen that DRAIN-2D is very sensitive to the number of degrees of freedom in the input structure, while EDAM2 is not as sensitive. The table also shows that the cost of running EDAM2 is slightly higher, by approximately 3 percent, than the cost of running the original EDAM.

As a final comment on execution cost, it is also noticed that if a convergence problem arises, the cost of running EDAM2 might triple, depending on the magnitude of oscillation occurring in the response period; a small oscillation takes fewer extra iterations to converge, and will result in a smaller increase in execution cost.
7.0 CONCLUSIONS

A method, based on elastic modal analysis, of analysing the non-linear response of structures to earthquake excitation has been presented. Since the method is capable of taking into account the redistribution of forces due to reduction in member stiffness, it is able to predict the pattern of damage that a planar structure would undergo during earthquake excitation.

Analyses of three different test structures, including two structural frames and a shear-wall frame, using the present method have shown good agreement when compared with the results obtained from the time-step analysis program DRAIN-2D. In all cases, the method is able to identify the weak spots and predict the deflected shape of the structure. A test frame with adrupt change in geometry along its height does not cause any particular problem in the method. It is also found that the method tends to work better when the damage on the structure is small and when damage occurs mainly in beams. Although it shows a certain weakness in predicting the magnitude of damage at the base of a structure, it is very good in the prediction of base shear. However, there are also some other weaknesses in the method that deserve special attention. Firstly, results determined using the present method are very sensitive to the shape of
the smooth spectrum, i.e., a slight change in spectral acceleration will cause significant change to the end results. Secondly, oscillations in spectral acceleration between iterations will create convergence problems. Finally, structures with weak columns will also cause convergence problems; hence, it is recommended that the method not be used for this type of structure.

In spite of these weaknesses, the proposed method does provide an efficient way of performing seismic analysis. It is relatively inexpensive to use, and yet, the results produced are comparable to those of an inelastic time-step analysis. It is also a practical substitute for elastic modal analysis in the design of medium size structures.
REFERENCES


10. Beaufait, F.W.  


12. Kanaan, A.E. And Graham, H.P.  

13. Jennings, P.C., Housner, G.W., and Tsai, N.C.  
'Simulated Earthquake Motions', California Institute of Technology, Pasadena, April 1968.

14. Murphy, L.M.  
\[ EI \]

\[ \frac{L}{L} = 1 \]

Table 5.1 Values for the effective damage ratio, \( \mu_e \)
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<th>3</th>
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<td>FUNDAMENTAL PERIOD (sec.)</td>
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<td>0.508</td>
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<td>FLOOR WEIGHT (kip)</td>
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<td>LEVEL 2-4</td>
<td>1460.0</td>
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</tr>
<tr>
<td></td>
<td>LEVEL 4-6</td>
<td>1460.0</td>
<td>340.6</td>
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<td></td>
<td>LEVEL 6-7</td>
<td>1410.0</td>
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<td></td>
<td>YOUNG'S MODULUS (ksi)</td>
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<td>3300</td>
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<td>EXT.</td>
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<td>1279</td>
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<td></td>
<td>LEVEL 2-7</td>
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<td></td>
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<td>LEVEL 2-7</td>
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Table 6.1 Structural properties for all test frames
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<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
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<th>3'</th>
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<td>0.3</td>
<td>0.3</td>
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<td>0.3</td>
<td>0.3</td>
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<tr>
<td>STRAIN HARDENING (%)</td>
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<td>0.075</td>
<td>1.50</td>
<td>0.075</td>
<td>1.50</td>
<td>1.50</td>
<td>0.075</td>
<td>0.075</td>
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<tr>
<td>RESPONSE PERIOD (sec.)</td>
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<td>0.892</td>
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<td>0.689</td>
<td>1.180</td>
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<td>0.091</td>
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<td>BASE SHEAR (%)</td>
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<td>4.6</td>
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<td>4.3</td>
<td>4.8</td>
<td>3.6</td>
<td>5.6</td>
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<td>DISPLACEMENT (%)</td>
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<td>20.8</td>
<td>30.3</td>
<td>8.7</td>
<td>14.0</td>
<td>30.4</td>
<td>27.4</td>
<td>47.3</td>
<td>42.7</td>
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<td>DAMAGE RATIO (abs.)</td>
<td>1.67</td>
<td>1.50</td>
<td>1.44</td>
<td>0.67</td>
<td>0.8</td>
<td>1.64</td>
<td>1.82</td>
<td>3.80</td>
<td>2.62</td>
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Table 6.2 Summary of results
<table>
<thead>
<tr>
<th>TEST FRAME NO.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>1'</th>
<th>2'</th>
<th>3'</th>
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<tr>
<td>NUMBER OF MODES</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td>4</td>
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<tr>
<td>NUMBER OF DEGREES OF FREEDOM</td>
<td>133</td>
<td>91</td>
<td>109</td>
<td>133</td>
<td>91</td>
<td>91</td>
<td>133</td>
<td>91</td>
<td>109</td>
</tr>
<tr>
<td>HALF BANDWIDTH OF STIFFNESS MATRIX</td>
<td>38</td>
<td>26</td>
<td>38</td>
<td>38</td>
<td>26</td>
<td>26</td>
<td>38</td>
<td>26</td>
<td>38</td>
</tr>
<tr>
<td>NO. OF ITERATIONS (EDAM, EDAM2)</td>
<td>8</td>
<td>10</td>
<td>7</td>
<td>9</td>
<td>6</td>
<td>7</td>
<td>8</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>NO. OF TIME STEPS (DRAIN-2D)</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>1500</td>
<td>480</td>
<td>400</td>
<td>400</td>
<td>400</td>
<td>400</td>
</tr>
<tr>
<td>EXECUTION TIME : (CPU sec.)</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>9.22</td>
<td>5.80</td>
<td>8.20</td>
</tr>
<tr>
<td>EDAM</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>EDAM2</td>
<td>9.51</td>
<td>6.57</td>
<td>6.64</td>
<td>10.58</td>
<td>4.09</td>
<td>4.70</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>DRAIN-2D</td>
<td>48.90</td>
<td>26.41</td>
<td>33.84</td>
<td>159.05</td>
<td>32.97</td>
<td>26.55</td>
<td>48.90</td>
<td>26.41</td>
<td>33.84</td>
</tr>
</tbody>
</table>

Table 6.3 Summary of program execution cost
Fig. 2.1 Diagrams for defining ductility ratio and damage ratio
MAIN PROGRAM

- read in program specifications, e.g. maximum number of iterations, initial damping, maximum acceleration, title etc.

- read in and organize structural information
- calculate number of D.O.F. and half bandwidth
- initialize damage ratios to 1.0

- set up mass matrix

- set up stiffness matrix

- check the condition of the stiffness matrix

- compute modal periods and mode shapes

- perform modal analysis:
  - determine smeared damping factors
  - calculate RSS forces and displacements
  - modified damage ratios
  (detail refer to Fig. 2.3)

Fig. 2.2 Flowchart for the main program
START

calculate modal participation factors

(calculate smeared)
damping factors

set KK = 1

mode no. = 1

SPECTR

calculate modal forces

SDFBAN

calculate displacements for each mode

DSBAND

sum the square of displacements

next mode

FORCE

Completed for all modes ?

no

yes

Is KK=2 ?

no

calculate (RSS forces)

set KK = 2

yes

compute RSS displacements

DAMOD

• modify damage ratios
• calculate member damping factors
• prepare for converge check
• compute RSS forces

RETURN

Fig. 2.3 Flowchart for the main subroutine - MOD3
Fig. 3.1 A smoothed response spectrum with all typical branches

Fig. 3.2 An example to show the fluctuation in response period vs. iteration number
Fig. 3.3 Graph showing the progress of each iteration plotted with the response spectrum.
Fig. 3.4 A plot of spectral acceleration vs. period showing the capacity and demand curve (Freeman's method)
Fig. 3.5. Results for the fourth and fifth iteration

(normalized to 0.2g)

\[ \beta_{S1} = 0.05267 \]

\[ \beta_{S1} = 0.06362 \]
Main Subroutine - MOD3

START

calculate modal participation factors

set KK = 1

mode no. = 1

Is mode no. = 1 ?

yes

STACHK

no

yes

SPECTR

Is LOCK = 0 ?

no

no

calculate modal forces

SDFBAN

DSBAND

sum the square of displacements

FORCE

Completed for all modes ?

no

yes

Is KK=2 ?

yes

compute RSS displacements

no

set KK = 2

DAMOD

RETURN

- detect oscillations in vibration period
- search for solution

Fig. 3.8 A revised flowchart for subroutine MOD3 including a new subroutine STACHK
Search Routine - STACHK

START

Is I COUNT ≥ 4 ?

no

RETURN

Is LOCK > 0 ?

no

Is KK = 2 ?

yes

store Sa (@ 2% damping)

store current period

compare it with previous two periods

Is there oscillation in period ?

no

RETURN

Is I COUNT ≥ 4 ?

yes

set upper and lower bound Sa (@ 2% damping)

set IFLAG = 0

modify Sa with appropriate damping

set LOCK = 1

Is IFLAG = 1 & KK = 2 ?

no

RETURN

check convergence:
compare Sa with Sa from spectrum

calculate the difference between two Sa's (SADIF)

PRINT trial no. in binary search & SADIF

PRINT calculate Sa actual Sa & SADIF

Is LOCK = 2 ?

no

Is difference in Sa < limit ?

no

adj ust lower or upper bound Sa (@ 2% damping)

Yes

set IFLAG = 0

set LOCK = 2

RETURN

normal procedure

search procedure

LOCK:
0 = follow normal routine
1 = follow search routine
2 = search completed

IFLAG:
0 = normal iteration
1 = last iteration

ICOUNT : iteration no.

Fig. 3.9 Flowchart for the solution searching routine - STACHK
Fig. 3.10 Graph showing the progress of the solution searching routine
Fig. 4.1  (a) Single-degree of freedom system
(b) Moment vs. rotation diagram
(c) Viscous damping force vs. displacement
Equation (4.10)  
(with 2% strain hardening)

Equation (4.11)

Equation (2.5)

Fig. 4.2 A plot of substitute damping factor vs. damage ratio
\[ k_a = \frac{AE}{L^3} \begin{bmatrix} x^2 & xy & 0 \\ xy & y^2 & 0 \\ 0 & 0 & 0 \end{bmatrix} \]

\[ k_b = \frac{12 EI}{L^5(1+h)\mu} \begin{bmatrix} y^2 & xy & -xL^2/2 \\ -xy & x^2 & L^2(1+h/4)/3 \\ -yL^2/2 & xL^2/2 & yL^2/2 \end{bmatrix} \]

(a) Axial stiffness matrix

(b) Bending stiffness matrix

Fig. 5.1 (a) Axial stiffness matrix

(b) Bending stiffness matrix
\[ k_r = \frac{12 EI}{\mu L^4} \begin{bmatrix} 0 & 0 & \text{SYMMETRICAL} \\ -ay & ax & aL(a+L) \\ 0 & 0 & ay \\ 0 & 0 & -ax \\ -by & bx & abL+L^2(a+b)/2 \\ by & -bx & bL^2+b^2L \end{bmatrix} \]

Fig. 5.1 (c) Rigid arm stiffness matrix

Fig. 5.2 A typical member with two hinges
Fig. 5.3 Diagrams for the conjugate beam method
Fig. 5.4 Different components of the distributed elastic load
Fig. 5.5 A plot of the effective damage ratio vs. $\mu_A$ and $\mu_B$
Fig. 5.6 Stiffness matrix with bending stiffnesses written in terms of $k_{33}$, $k_{66}$ and $k_{63}$
\[
\begin{bmatrix}
3 & 6 & 2 & 5 & 1 & 4 \\
k_{33} & k_{66} & & & & \\
k_{63} & & & & & \\
k_{xk_{23}} & C_xk_{26} & C_x^2k_{22} + C_y^2k_{11} & & & \\
k_{xk_{53}} & C_xk_{56} & C_x^2k_{52} + C_y^2k_{41} & C_x^2k_{55} + C_y^2k_{44} & & \\
-C_yk_{23} & -C_yk_{26} & C_xC_yk_{11} - C_xC_yk_{22} & C_xC_yk_{41} - C_xC_yk_{52} & C_x^2k_{11} + C_y^2k_{22} & \\
-C_yk_{53} & -C_yk_{56} & C_xC_yk_{41} - C_xC_yk_{52} & C_xC_yk_{44} - C_xC_yk_{55} & C_x^2k_{41} + C_y^2k_{52} & C_x^2k_{44} + C_y^2k_{55} \\
\end{bmatrix}
\]

**SYMMETRICAL**

\[C_x = \frac{x}{L} \quad C_y = \frac{y}{L}\]

Fig. 5.7 A transformed stiffness matrix

\[
\begin{bmatrix}
\mu_6(y^2) & \mu_6(x^2) & & & & \\
\mu_6(-xy) & \mu_6(x^2) & & & & \\
\mu_4(-yL^2/2) & \mu_4(xL^2/2) & \mu_1(L^4/3) & & & \\
\mu_6(-y^2) & \mu_6(xy) & \mu_4(yL^2/2) & \mu_6(y^2) & & \\
\mu_6(xy) & \mu_6(-x^2) & \mu_4(-xL^2/2) & \mu_6(-xy) & \mu_6(x^2) & \\
\mu_5(-yL^2/2) & \mu_5(xL^2/2) & \mu_3(L^4/6) & \mu_5(yL^2/2) & \mu_5(-xL^2/2) & \mu_2(3L^4) \\
\end{bmatrix}
\]

**SYMMETRICAL**

\[k_b = \frac{12EI}{L^5}\]

Fig. 5.8 Bending stiffness matrix with six modification factors
FIG. 5.9 A revised flowchart for the main program

- DAMCAL - calculate factors from Eqn. (5.13) to Eqn. (5.18)
- BUILD - assemble bending stiffness matrix in Fig. 5.8
START

- calculate modal participation factors

set KK = 1

mode no. = 1

Is mode no. = 1?

STACHK

yes

- detect oscillations in vibration period
- search for solution

no

Is LOCK = 0?

no

SPECTR

calculate modal forces

SDFBAN

DSBAND

sum the square of displacements

next mode

FORCE

DAMCAL

Completed for all modes?

no

yes

compute RSS displacements

Is KK=2?

no

set KK = 2

.DAMOD

RETURN

- DAMCAL - calculate factors from Eqn. (5.13) to (5.18)
- FORCE - replace Eqn. (2.6) with Eqn. (5.22)
- replace Eqn. (4.10) with Eqn. (5.23)
- modify two damage ratios for each member

Fig. 5.10 A revised flowchart for subroutine MOD3 including all changes
C.I.T. SIMULATED EARTHQUAKE: TYPE C2
(normalized to 0.2g)

SPECTRAL ACCELERATION Sa (g)

PERIOD (SEC.)

Fig. 6.1 C.I.T. simulated earthquake type C2 spectrum
SAN FERNANDO EARTHQUAKE  8244 Orion Ave.
Holiday Inn, ground floor

Fig. 6.2 San Fernando S90W spectrum
Fig. 6.3 Dimensions and yield moments for Test Frame No. 1
RSS BASE SHEAR (EDAM2) 2096.7 kips
MAX. BASE SHEAR (DRAIN-2D) 2259.0 kips

### Damage Ratios

<table>
<thead>
<tr>
<th>Test Frame No. 1</th>
<th>Damage Ratios</th>
<th>EDAM2</th>
<th>DRAIN-2D</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.44 (0.35)</td>
<td>0.64 (0.55)</td>
<td>0.57 (0.50)</td>
<td>0.58 (0.50)</td>
</tr>
<tr>
<td>0.37 (0.29)</td>
<td>0.72 (0.60)</td>
<td>0.78 (0.68)</td>
<td>0.77 (0.67)</td>
</tr>
<tr>
<td>0.97 (0.66)</td>
<td>1.05 (0.77)</td>
<td>1.03 (0.74)</td>
<td>1.02 (0.74)</td>
</tr>
<tr>
<td>0.61 (0.39)</td>
<td>0.84 (0.57)</td>
<td>0.84 (0.60)</td>
<td>0.84 (0.60)</td>
</tr>
<tr>
<td>1.78 (0.91)</td>
<td>1.86 (0.93)</td>
<td>1.86 (0.92)</td>
<td>1.86 (0.92)</td>
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<tr>
<td>0.57 (0.53)</td>
<td>0.70 (0.64)</td>
<td>0.68 (0.64)</td>
<td>0.68 (0.64)</td>
</tr>
<tr>
<td>2.94 (2.37)</td>
<td>3.11 (2.46)</td>
<td>3.11 (2.46)</td>
<td>3.11 (2.46)</td>
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<tr>
<td>0.56 (0.72)</td>
<td>0.64 (0.84)</td>
<td>0.67 (0.67)</td>
<td>0.67 (0.67)</td>
</tr>
<tr>
<td>3.24 (3.56)</td>
<td>3.37 (3.72)</td>
<td>3.37 (3.72)</td>
<td>3.37 (3.72)</td>
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<tr>
<td>0.47 (0.64)</td>
<td>0.55 (0.68)</td>
<td>0.53 (0.66)</td>
<td>0.53 (0.66)</td>
</tr>
<tr>
<td>3.61 (4.56)</td>
<td>3.79 (4.85)</td>
<td>3.79 (4.85)</td>
<td>3.79 (4.85)</td>
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<tr>
<td>0.49 (0.57)</td>
<td>0.62 (0.69)</td>
<td>0.57 (0.65)</td>
<td>0.57 (0.65)</td>
</tr>
<tr>
<td>4.83 (6.32)</td>
<td>5.09 (6.77)</td>
<td>5.09 (6.76)</td>
<td>5.09 (6.76)</td>
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<tr>
<td>0.57 (0.67)</td>
<td>0.75 (0.88)</td>
<td>0.74 (0.87)</td>
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**Fig. 6.4** Test Frame No. 1 - maximum base shear, damage ratios and horizontal displacements
Fig. 6.5 Beam damage ratio on Test Frame No. 1
Fig. 6.6 Dimensions and yield moments for Test Frame No. 2
RSS BASE SHEAR (EDAM2)  635.9 kips

MAX. BASE SHEAR (DRAIN-2D)  655.6 kips

C.I.T. - TYPE C2 SIMULATED EARTHQUAKE
MAXIMUM GROUND ACCELERATION 0.3g

Fig. 6.7 Test Frame No. 2 - maximum base shear, damage ratios and horizontal displacements
Fig. 6.8 Beam damage ratio on Test Frame No. 2
Fig. 6.9 Dimensions and yield moments for Test Frame No. 3
### RSS Base Shear (EDAM2)
- **2162.6 kips**

### MAX. Base Shear (DRAIN-2D)
- **2031.6 kips**

### C.I.T. - TYPE C2 SIMULATED EARTHQUAKE
- **MAXIMUM GROUND ACCELERATION 0.3g**

<table>
<thead>
<tr>
<th>C.I.T. Type</th>
<th>MAX. BASE SHEAR</th>
<th>DAMAGE RATIOS</th>
<th>EDAM2</th>
</tr>
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<tbody>
<tr>
<td>0.77</td>
<td>0.70</td>
<td>0.74</td>
<td>0.77</td>
</tr>
<tr>
<td>1.56</td>
<td>1.28</td>
<td>1.33</td>
<td>1.25</td>
</tr>
<tr>
<td>2.22</td>
<td>2.02</td>
<td>2.02</td>
<td>2.15</td>
</tr>
<tr>
<td>0.92</td>
<td>0.95</td>
<td>0.95</td>
<td>0.95</td>
</tr>
<tr>
<td>2.80</td>
<td>2.44</td>
<td>2.45</td>
<td>2.65</td>
</tr>
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<td>0.86</td>
<td>0.81</td>
<td>0.82</td>
<td>0.82</td>
</tr>
<tr>
<td>1.81</td>
<td>1.53</td>
<td>1.50</td>
<td>2.35</td>
</tr>
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<td>0.88</td>
<td>0.88</td>
<td>0.97</td>
<td>2.04</td>
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<tr>
<td>0.97</td>
<td>0.97</td>
<td>0.55</td>
<td>0.51</td>
</tr>
<tr>
<td>2.47</td>
<td>2.54</td>
<td>2.58</td>
<td>2.59</td>
</tr>
<tr>
<td>0.72</td>
<td>0.70</td>
<td>0.67</td>
<td>0.66</td>
</tr>
<tr>
<td>0.61</td>
<td>0.69</td>
<td>0.63</td>
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<tr>
<td>3.91</td>
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<td>4.13</td>
<td>4.14</td>
</tr>
<tr>
<td>0.79</td>
<td>0.80</td>
<td>0.79</td>
<td>0.79</td>
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</tbody>
</table>

*Fig. 6.10 Test Frame No. 3 - maximum base shear and damage ratios*
Fig. 6.11(a) Beam damage ratio on Test Frame No. 3
Fig. 6.11(a) Beam damage ratio on Test Frame No. 3 (cont'd) (b) Horizontal displacements
RSS BASE SHEAR (EDAM2)  2117.9 kips  

MAX. BASE SHEAR (DRAIN-2D)  2214.4 kips

San Fernando S90W Earthquake  
Maximum Ground Acceleration 0.2g

<table>
<thead>
<tr>
<th>Horizontal Displacement (in.)</th>
<th>Damage Ratios (EDAM2)</th>
<th>Damage Ratios (DRAIN-2D)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.29 (0.32)</td>
<td>0.51 (0.50)</td>
<td>0.44 (0.44)</td>
</tr>
<tr>
<td>0.24 (0.26)</td>
<td>0.52 (0.54)</td>
<td>0.61 (0.61)</td>
</tr>
<tr>
<td>0.47 (0.47)</td>
<td>0.69 (0.68)</td>
<td>0.70 (0.69)</td>
</tr>
<tr>
<td>1.49 (1.31)</td>
<td>1.55 (1.28)</td>
<td>1.55 (1.28)</td>
</tr>
<tr>
<td>0.62 (0.61)</td>
<td>0.75 (0.73)</td>
<td>0.72 (0.72)</td>
</tr>
<tr>
<td>3.30 (2.70)</td>
<td>3.50 (2.84)</td>
<td>3.50 (2.84)</td>
</tr>
<tr>
<td>0.69 (0.71)</td>
<td>0.81 (0.83)</td>
<td>0.79 (0.80)</td>
</tr>
<tr>
<td>4.21 (3.59)</td>
<td>4.43 (3.76)</td>
<td>4.43 (3.76)</td>
</tr>
<tr>
<td>0.54 (0.61)</td>
<td>0.61 (0.67)</td>
<td>0.59 (0.65)</td>
</tr>
<tr>
<td>4.79 (4.34)</td>
<td>5.09 (4.60)</td>
<td>5.09 (4.60)</td>
</tr>
<tr>
<td>0.58 (0.57)</td>
<td>0.70 (0.69)</td>
<td>0.66 (0.65)</td>
</tr>
<tr>
<td>6.08 (5.70)</td>
<td>6.46 (6.07)</td>
<td>6.46 (6.06)</td>
</tr>
<tr>
<td>0.62 (0.63)</td>
<td>0.82 (0.83)</td>
<td>0.81 (0.82)</td>
</tr>
</tbody>
</table>

Fig. 6.12 Test Frame No. 4 - maximum base shear, damage ratios and horizontal displacements
Fig. 6.13 Beam damage ratio on Test Frame No. 4
Fig. 6.14 Dimensions and yield moments for Test Frame No. 5
### RSS Base Shear (EDAM2)
- 599.0 kips

### Max. Base Shear (DRAIN-2D)
- 599.1 kips

**C.I.T. - Type C2 Simulated Earthquake**
- Maximum Ground Acceleration: 0.2g

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>EDAM2</th>
<th>DRAIN-2D</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.82</td>
<td>0.82</td>
</tr>
<tr>
<td>1.0</td>
<td>0.83</td>
<td>0.84</td>
</tr>
<tr>
<td>2.0</td>
<td>0.84</td>
<td>0.84</td>
</tr>
<tr>
<td>3.0</td>
<td>0.86</td>
<td>0.84</td>
</tr>
<tr>
<td>4.0</td>
<td>0.87</td>
<td>0.85</td>
</tr>
<tr>
<td>5.0</td>
<td>0.88</td>
<td>0.85</td>
</tr>
<tr>
<td>6.0</td>
<td>0.89</td>
<td>0.88</td>
</tr>
<tr>
<td>7.0</td>
<td>0.90</td>
<td>0.88</td>
</tr>
</tbody>
</table>

**Fig. 6.15 Test Frame No. 5 - maximum base shear, damage ratios and horizontal displacements**
Fig. 6.16 Column damage ratio on Test Frame No. 5
RSS BASE SHEAR (EDAM2)  646.9 kips

MAX. BASE SHEAR (DRAIN-2D)  674.7 kips

<table>
<thead>
<tr>
<th>Floor Level</th>
<th>EDAM2 (kips)</th>
<th>DRAIN-2D (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.63 (2.29)</td>
<td>1.17 (1.70)</td>
</tr>
<tr>
<td>2</td>
<td>2.08 (2.44)</td>
<td>1.82 (2.16)</td>
</tr>
<tr>
<td>3</td>
<td>2.15 (2.50)</td>
<td>2.01 (2.41)</td>
</tr>
<tr>
<td>4</td>
<td>2.23 (2.57)</td>
<td>2.26 (2.63)</td>
</tr>
<tr>
<td>5</td>
<td>3.54 (4.78)</td>
<td>2.42 (3.91)</td>
</tr>
</tbody>
</table>

C.I.T.- TYPE C2 SIMULATED EARTHQUAKE
MAXIMUM GROUND ACCELERATION  0.3g

Test Frame No. 6 - maximum base shear, damage ratios and horizontal displacements

Fig. 6.17
Fig. 6.18  Beam damage ratio on Test Frame No. 6
Fig. 6.19 A comparison of beam damage ratios from EDAM, EDAM2 and DRAIN-2D - Test Frame No. 1
TEST FRAME NO. 2 (STARTING FROM THE LEFT)

Fig. 6.20 A comparison of beam damage ratios from EDAM, EDAM2 and DRAIN-2D - Test Frame No. 2
Fig. 6.21 A comparison of beam damage ratios from EDAM, EDAM2 and DRAIN-2D - Test Frame No. 3
Fig. 6.21 A comparison of beam damage ratios from EDAM, EDAM2 and DRAIN-2D - Test Frame No. 3 (cont'd)
APPENDIX A USER'S MANUAL

ELASTIC OR DAMAGE AFFECTED MODAL ANALYSIS VER. 2.0

Program Name : EDAM2
Programmed by : Lawrence H.Y. Hui
Andrew W.F. Metten
Sumio Yoshida
The University of British Columbia, 1984

DISCLAIMER

THE CIVIL ENGINEERING DEPARTMENT, FACULTY AND STAFF DO NOT GUARANTEE NOR IMPLY THE ACCURACY OR RELIABILITY OF THIS PROGRAM OR RELATED DOCUMENTATION. AS SUCH, THEY CAN NOT BE HELD RESPONSIBLE FOR INCORRECT RESULTS OR DAMAGES RESULTING FROM THE USE OF THIS PROGRAM. IT IS THE RESPONSIBILITY OF THE USER TO DETERMINE THE USEFULNESS AND TECHNICAL ACCURACY OF THIS PROGRAM IN HIS OR HER OWN ENVIRONMENT.

THIS PROGRAM MAY NOT BE SOLD TO A THIRD PARTY.
PURPOSE

The purpose of the program is to determine the dynamic response of inelastic two-dimensional frame or shear-wall structures using a modified modal analysis. Results of the analysis will include the prediction of a damage pattern in terms of damage ratios, also the displacements and forces of the modelled structure under a specific earthquake excitation.

The program can perform ordinary modal analysis on any two-dimensional structures as well.

PROGRAM RESTRICTIONS

The following restrictions apply to BOTH elastic and damage affected analysis.

1. The system can be analysed in one vertical plane.

2. Non-structural components are to be designed such that they do not affect the response of the system as modelled.

3. The program cannot handle more than one type of material directly. Should the user desire to test a structure that contains more than one type of material, the members constructed of the second material type should have the area and inertia multiplied by the modular ratio (E for type 1 divided by E for type 2).

4. The program applies the same acceleration to both horizontal and vertical masses. Therefore vertical masses should not be included; should masses be attached to some of the nodes then only mode shapes and frequencies will be computed correctly. This restriction limits the program to the analysis of structures for which vertical acceleration of the nodes is not a significant factor. For most structures with masses on vertical column lines this restriction will not be a limiting consideration.

5. The structure must comply with the dimensioning requirements of EDAM2.
The following restrictions apply ONLY to damage affected analysis.

1. The materials used for the construction of the structure must be concrete. The development of the stiffness degradation and damping formula was done completely on concrete members and the research does not apply to steel or other non-concrete materials.

2. The members must be designed such that they can withstand the damage ratios imposed without undergoing brittle failure.

3. The members are assumed to be symmetric and have the same moment capacity under both positive and negative bending moments.

4. The fundamental period should be such that it places the structure on a segment of the spectrum which causes a decrease in the spectral acceleration when the period of the structure increases.

5. The structure should be so designed that majority of the damage will occur at the beams while only minor damage will occur at the columns.

The program has been developed for Amdahl 470 V/8-II, which is IBM 360/370 compatible, operating under the Michigan Terminal System (MTS) of the University of British Columbia. In order to run the program in another institution which is using a different computing system, the Eigenvalue solver and the matrix routines in the program might have to be replaced. To obtain terminal output of intermediate results during program execution, some Fortran commands, which are used only in MTS, have to be changed also.
CAPACITY LIMITATIONS

The maximum dimensions of the structure are:

- 150 members
- 150 joints
- 100 assigned masses
- 10 modes

\[(\text{total degrees of freedom}) \times (\text{half bandwidth}) \leq 8000\]

These dimensions can be changed by adjusting the DIMENSION statements in the program, as well as the limits stated in the main program.

EXECUTION TIME

The execution time for the program depends on the following items:

1. the number of degrees of freedom in the structure
2. the half-bandwidth of the stiffness matrix
3. the number of lumped masses
4. the number of modes included in the analysis
5. the number of iterations required to complete the analysis
INPUT DATA

I. PROBLEM INITIATION:
INELAS, NMODES, NPRINT, ISPEC, AMAX, DAMPIN
FORMAT (4I5, 2F10.5) - ONE CARD

INELAS: Maximum number of iteration for inelastic analysis (NOTE 1)
0 = Elastic modal analysis

NMODES: Number of modes (≤ 10) to be included in the analysis (NOTE 2)

NPRINT: Number of modes for which displacements and forces will be printed

ISPEC: Input spectrum type -
1 = Spectrum 'A' from Shibata and Sozen
2 = Spectrum 'B' from Yoshida
3 = Spectrum 'C' from Yoshida
4 = National Building Code spectrum
5 = San Fernando earthquake S90W spectrum
6 = C.I.T. Simulated earthquake type C2 spectrum

AMAX: Maximum ground acceleration (g)

DAMPIN: Elastic or initial damping expressed as a fraction of critical damping

II. TITLE:
TITLE
FORMAT (20A4) - ONE CARD

Maximum length is 80 characters

III. GENERAL STRUCTURAL INFORMATION:
NRJ, NRM, E, G, HARD
FORMAT (2I5, 2F10.3, F10.8) - ONE CARD

NRJ: Number of joints in the structure

NRM: Number of members in the structure

E: Young's modulus (ksi)

G: Shear modulus (ksi)
0 = neglect shear deflection

HARD: Strain hardening ratio, as a proportion of initial stiffness
IV. JOINT INFORMATION:
JN,NDX,NDY,NDR,X,Y
FORMAT (4I5,2F10.3) - ONE CARD/JOINT

JN: Node number

NDX: 0 = Joint is fixed in the x-direction
1 = Joint is free to move in the x-direction
N = Same motion in x-direction as node N

NDY: 0 = Joint is fixed in the y-direction
1 = Joint is free to move in the y-direction
N = Same motion in y-direction as node N

NDR: 0 = Joint is not allow to rotate
1 = Joint is free to rotate
N = Same rotation as node N

X: X-coordinate of the joint (ft.)

Y: Y-coordinate of the joint (ft.)

V. MEMBER PROPERTIES:
MN,JNL,JNG,KL,KG,AREA,CRMOM,AV,BMCAP,EXTL,EXTG
FORMAT (5I5,F8.2,F12.3,2F10.3,2F6.3) - ONE CARD/MEMBER

MN: Member number

JNL: The lesser joint number

JNG: The greater joint number (NOTE 3)

KL: 1 = Member is fixed at the lesser joint
0 = Member is pinned at the lesser joint

KG: 1 = Member is fixed at the greater joint
0 = Member is pinned at the greater joint

AREA: Cross sectional area of the member (in^2)

CRMOM: Moment of inertia of the member (in^4)

AV: Shear area of the member (in^2) (NOTE 4)

BMCAP: Yield moment of the member (kip-ft.)

EXTG: Rigid extension on the lesser joint end of member (ft.)

EXTL: Rigid extension on the greater joint end of member (ft.) (NOTE 5)
VI. NUMBER OF JOINTS WITH LUMPED MASSES:
NMASS
FORMAT (I5) - ONE CARD

NMASS: Number of nodes to which a weight is attached

VII. SPECIFICATION FOR LUMPED MASSES:
JN,WTX,WTY,WTR
FORMAT (I5,3F10.0) - ONE CARD/JOINT WITH MASS

JN: Joint number
WTX: Weight in x-direction (kip)
WTY: Weight in y-direction (kip)
WTR: Rotational weight
(see Point 4 in program restrictions)

NOTES

1. INELAS is the maximum number of iterations that the program will perform if convergence is not attained within this limit. A value of 40 is usually sufficient.

2. If there are less than NMODES degrees of freedom to which masses are attached, then NMODES will be set equal to the number of degrees of freedom to which masses are attached.

3. The ordering of the joint numbers will not affect the results produced. For every member there is a joint numbering that will cause either x or y displacements to be printed as a negative number. The printing of negative displacements of the member should not disturb the user.

4. Since testing is incomplete on the program's ability to handle shear deflections, a zero value should be assigned to AV at this stage so that shear deflections will not be computed.

5. At this stage of program development, the rigid arms can only be attached to horizontal members. The attachment of rigid arms to non-horizontal members will result in the printing of an error message.
OUTPUT UNITS

UNIT 6  This file is for data output from intermediate iterations from inelastic analysis. Nothing of importance is written in output unit 6, therefore printed output for this file is generally not required. Also, printing of data from unit 6 may result in a very lengthy output.

UNIT 7  Input data and results for either elastic or inelastic analysis are printed through output unit 7. It also contains the error message when problem arises. Note that for elastic analysis, the input moment capacities have little purpose neither will the output damage ratios.

UNIT 99  Unit 99 is assigned by a Fortran command statement inside the program to I/O unit SPRINT, from which essential results obtained after each iteration are printed on the terminal during program execution. In the process, the user should be able to determine whether convergence problem has occurred.

OPERATING INSTRUCTION

The following statement can be used to compile the source program

$RUN *FTN SCARDS=EDAM2

The object code of the program will be store in a temporary file -LOAD and can be copied to a permanent disk file for later use if necessary. The program can be executed by issuing the following command

$RUN -LOAD 5=INPUT 6=-6 7=-7

where INPUT is the input file, and -6, -7 are temporary output files.

If output from unit 6 is not required by the user, a dummy file should be assigned to the unit so that the program can run more efficiently and also save the cost of using more virtual memory. This can be done by the following command

$RUN -LOAD 5=INPUT 6=*DUMMY* 7=-7
Fig. B.1  Illustrative example
I. **SAMPLE INPUT**

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<tr>
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<th>4</th>
<th>1</th>
<th>6</th>
<th>0.20000</th>
<th>0.02000</th>
</tr>
</thead>
<tbody>
<tr>
<td>SAMPLE</td>
<td>INPUT/OUTPUT</td>
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<td>23</td>
<td>3300.0000</td>
<td>0.0</td>
</tr>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>18.0000</td>
<td>0.0</td>
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<td>3</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>36.0000</td>
<td>0.0</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>54.0000</td>
<td>0.0</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>81.0000</td>
<td>0.0</td>
</tr>
<tr>
<td>6</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.0</td>
<td>10.0000</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>1</td>
<td>1</td>
<td>18.0000</td>
<td>10.0000</td>
</tr>
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<td>8</td>
<td>6</td>
<td>1</td>
<td>1</td>
<td>36.0000</td>
<td>10.0000</td>
</tr>
<tr>
<td>9</td>
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<td>1</td>
<td>1</td>
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<td>10.0000</td>
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<td>10</td>
<td>6</td>
<td>1</td>
<td>1</td>
<td>81.0000</td>
<td>10.0000</td>
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<td>11</td>
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<td>1</td>
<td>1</td>
<td>0.0</td>
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<td>1</td>
<td>1</td>
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<td>20.0000</td>
</tr>
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<td>20.0000</td>
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<td>14</td>
<td>11</td>
<td>1</td>
<td>1</td>
<td>54.0000</td>
<td>20.0000</td>
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<td>1</td>
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<td>20.0000</td>
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<td>16</td>
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<td>1</td>
<td>1</td>
<td>36.0000</td>
<td>30.0000</td>
</tr>
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<td>81.0000</td>
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<td>1</td>
<td>81.0000</td>
<td>30.0000</td>
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<td>21</td>
<td>16</td>
<td>1</td>
<td>1</td>
<td>81.0000</td>
<td>30.0000</td>
</tr>
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<td>22</td>
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<td>1</td>
<td>1</td>
<td>81.0000</td>
<td>30.0000</td>
</tr>
<tr>
<td>23</td>
<td>16</td>
<td>1</td>
<td>1</td>
<td>81.0000</td>
<td>30.0000</td>
</tr>
</tbody>
</table>
II. OUTPUT FROM TERMINAL

```
$RUN EDAM2RC 5=SAMPLE 6="DUMMY" 7=-7
Execution begins

<table>
<thead>
<tr>
<th>ITERATION NO.</th>
<th>NO. ABOVE CAPACITY</th>
<th>DAMDIF</th>
<th>S MATRIX RATIO</th>
<th>SMEARED DAMPING</th>
</tr>
</thead>
<tbody>
<tr>
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<td>17</td>
<td>0.803</td>
<td>0.757E+03</td>
<td>0.02000</td>
</tr>
<tr>
<td>2</td>
<td>5</td>
<td>0.107</td>
<td>0.762E+03</td>
<td>0.03545</td>
</tr>
<tr>
<td>3</td>
<td>9</td>
<td>0.078</td>
<td>0.761E+03</td>
<td>0.03823</td>
</tr>
<tr>
<td>4</td>
<td>9</td>
<td>0.059</td>
<td>0.760E+03</td>
<td>0.03913</td>
</tr>
<tr>
<td>5</td>
<td>9</td>
<td>0.057</td>
<td>0.760E+03</td>
<td>0.03963</td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>0.035</td>
<td>0.761E+03</td>
<td>0.03993</td>
</tr>
<tr>
<td>7</td>
<td>2</td>
<td>0.055</td>
<td>0.761E+03</td>
<td>0.04013</td>
</tr>
<tr>
<td>8</td>
<td>0</td>
<td>0.039</td>
<td>0.761E+03</td>
<td>0.04026</td>
</tr>
<tr>
<td>9</td>
<td>0</td>
<td>0.021</td>
<td>0.761E+03</td>
<td>0.04033</td>
</tr>
<tr>
<td>10</td>
<td>0</td>
<td>0.007</td>
<td>0.761E+03</td>
<td>0.04040</td>
</tr>
</tbody>
</table>

APPROX. NUMBER OF SIG. FIGURES IN EIGENVALUE IN EIGENVECTOR

<table>
<thead>
<tr>
<th>EIGEN RESIDUAL APPROX. IN EIGENVALUE IN EIGENVECTOR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 3.477126E-32 15.0 7.5</td>
</tr>
<tr>
<td>2 1.138719E-29 15.0 7.5</td>
</tr>
<tr>
<td>3 7.389004E-31 15.0 7.5</td>
</tr>
</tbody>
</table>

* * * STURM SEQUENCE TEST INDICATES ALL 3 SMALLEST EIGENVALUES FOUND < PERIOD = 0.189 SA = 0.707 >

Execution terminated
```
III. OUTPUT FROM UNIT 7

*******PROGRAM OPTIONS*******
MAXIMUM NUMBER OF MODES IN ANALYSIS 4
INELASTIC ANALYSIS MAXIMUM ITERATIONS* 20
INITIAL DAMPING RATIO* 0.020
NUMBER OF MODES TO HAVE OUTPUT PRINTED* 1

SEISMIC INPUT

MAXIMUM ACCELERATION: 200 TIMES GRAVITY
CIT/SIMULATED EARTHQUAKE TYPE C-2 SPECTRUM
### Sample Input/Output

\[ E = 3300.0 \text{ ksi} \quad G = 0.0 \text{ ksi} \quad \text{Strain Hardening Ratio} = 0.02000 \]

<table>
<thead>
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</tr>
</thead>
<tbody>
<tr>
<td>( \text{JN} )</td>
</tr>
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</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
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</tr>
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</tr>
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<td>17</td>
</tr>
<tr>
<td>18</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Member Data</th>
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</thead>
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</tr>
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</tr>
<tr>
<td>2</td>
</tr>
<tr>
<td>3</td>
</tr>
<tr>
<td>4</td>
</tr>
<tr>
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<tr>
<td>12</td>
</tr>
<tr>
<td>13</td>
</tr>
<tr>
<td>14</td>
</tr>
</tbody>
</table>
NO. OF DEGREES OF FREEDOM OF STRUCTURE = 29
HALF BANDWIDTH OF STIFFNESS MATRIX = 22

NO. OF NODES WITH MASS = 3

<table>
<thead>
<tr>
<th>JN</th>
<th>X-MASS (KIPS)</th>
<th>Y-MASS (KIPS)</th>
<th>ROT. MASS (IN-KIPS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>330.000</td>
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<td>0.0</td>
</tr>
<tr>
<td>13</td>
<td>280.000</td>
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<td>0.0</td>
</tr>
<tr>
<td>17</td>
<td>220.000</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

MASS NO. DOF ASSIGNED MASS (KIP-SEC^2/FT)

<table>
<thead>
<tr>
<th>JN</th>
<th>DOF</th>
<th>ASSIGNED MASS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>10.24845</td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>8.69565</td>
</tr>
<tr>
<td>3</td>
<td>23</td>
<td>6.83230</td>
</tr>
</tbody>
</table>

INITIAL ELASTIC PERIOD

<table>
<thead>
<tr>
<th>MODES</th>
<th>EIGENVALUES (RAD/SEC)</th>
<th>NATURAL FREQUENCIES (CVCS/SEC)</th>
<th>PERIODS (SECS)</th>
<th>2 PERCENT DAMPING</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<td>36.4558</td>
<td>5.8022</td>
<td>0.1723</td>
</tr>
<tr>
<td>2</td>
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<td>199.3018</td>
<td>31.7200</td>
<td>0.0314</td>
</tr>
<tr>
<td>3</td>
<td>256996.5625</td>
<td>506.9482</td>
<td>80.6837</td>
<td>0.0124</td>
</tr>
</tbody>
</table>

INELASTIC RESULTS

DAMERR = 0.01

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<tr>
<th>ITERATION</th>
<th>NO. ABOVE DAMIF S MATRIX</th>
<th>SMEARED RATIO DAMPING</th>
<th>PERIOD</th>
<th>SA</th>
<th>DAMERR</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17</td>
<td>0.803</td>
<td>0.757E+03</td>
<td>0.02000</td>
<td>0.172</td>
</tr>
</tbody>
</table>
ITERATION NUMBER 11

ALL ELEMENTS OF MAIN DIAGONAL OF STIFFNESS MATRIX ARE POSITIVE DEFINITE
RATIO OF LARGEST TO SMALLEST DIAGONAL STIFFNESS MATRIX ELEMENT IS 0.76E+03

NO. OF MODES TO BE ANALYZED = 3

<table>
<thead>
<tr>
<th>MODE</th>
<th>EIGENVALUES</th>
<th>NATURAL FREQUENCIES</th>
<th>PERIODS</th>
<th>SA</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(RAD/SEC)</td>
<td>(CYS/SEC)</td>
<td>(SECS)</td>
<td>(2 PERCENT DAMPING)</td>
</tr>
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<td>33.2011</td>
<td>0.189</td>
<td>0.8870</td>
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<tr>
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<tr>
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<td>248227.5000</td>
<td>498.2244</td>
<td>0.0126</td>
<td>0.1809</td>
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</tbody>
</table>

MODAL PARTICIPATION FACTOR

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MODE SMEARED DAMPING RATIO

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### Mode Number 1 Modal Forces and Displacements

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### Root Mean Square Displacements

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<th>X-Disp(FT)</th>
<th>Y-Disp(FT)</th>
<th>Rotation(Rad)</th>
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ROOT MEAN SQUARE FORCES
RSS BASE SHEAR = 419.603 KIPS

NO. OF ITERATIONS = 11

BETA=0.800
BENDING MOMENT ERROR=0.050000
DAMAGE RATIO ERROR=0.010
PROGRAM DIMENSIONED FOR A MAXIMUM OF :-

- 150 MEMBERS
- 150 JOINTS
- 100 ASSIGNED MASSES
- 10 EIGENVALUES
- 300 UNKNOWNS
- (NUMBER OF UNKNOWNS)*(HALF BANDWIDTH) IS LESS THAN 8000

* NOTE - PRITZ IS A URCA: MATRIX LIBRARY SUBROUTINE
FOR SOLVING EIGENVALUES

VARIABLE DEFINITIONS:-

- KL,KG = JOINT TYPE - FIXED JOINT = 1
- PINNED JOINT = 0
- AREA = CROSS-SECTIONAL AREA
- CROHOM = MOMENT OF INERTIA OF CRACKED SECTION
- BMCP = BENDING MOMENT CAPACITY OF SECTION
- DAMAT = DAMAGE RATIO OF MEMBER
- ND = D.O.F. NO. IDENTIFIED BY JOINT NO.
- NOK,1) = K * (1,X-DOF). 2 (Y-DOF). 3 (R-DOF)
- NP = D.O.F. NO. IDENTIFIED BY MEMBER NO.
- NPI K,1) = K * DOF 1 TO 6 FOR STANDARD MEMBER
- XM = LENGTH OF FLEXIBLE PORTION OF BEAM IN X-DIRECTION
- YM = LENGTH OF FLEXIBLE PORTION OF BEAM IN Y-DIRECTION
- DM = TRUE LENGTH OF FLEXIBLE PORTION OF BEAM
- F = LOAD VECTOR
- EXTL,EXTG = LENGTH OF RIGID END
- TITLE = TITLE (80 CHARACTERS)
- SDAMP = STRUCTURAL DAMPING
- AV = SHEAR AREA
- DAMB = DAMAGE RATIO IN THE (1-1)TH ITERATION
- MDAT = D.O.F. NO. FOR MASSES IDENTIFIED BY MASS NO.
- AMASS = LUMPED MASS (IN UNITS OF WEIGHT) IDENTIFIED BY
- D.D.F. NO.
- EVAL = EIGENVALUE FOR EACH MODE
- EVEC = MODE SHAPE
- EVEC(I,1) = K * MASS NO.
- BETAM = SMEARED SUBSTITUTE DAMPING FOR EACH Mode
- LOCK = CONTROL DIFFERENT STAGE OF ITERATION. CORRESPOND TO
- SUBROUTINE STACHK WHICH IS USED TO STABILIZE CONVERGENCE
- LOCK = 0 - USUAL CONVERGENCE PROCEDURE
- 1 - BINARY SEARCH ROUTINE IN EFFECT
- 2 - PROGRAM CONVERGE
- OLTDN = STORE THE PERIOD OF THE LAST ITERATION
- OLDTA = STORE UPPER AND LOWER BOUND 5A VALUES

DOUBLE PRECISION STIFFNESS MATRIX

REAL*8 I 60000

DIMENSION KL(150),KG(150),AREA(150),CROHOM(150),BMCP(150),
ND(1,150),NP(6,150),XM(150),YM(150),DM(150),
F(100),EXTL(150),EXTG(150),TITLE(120),SDAMP(150),AV(150)
DIMENSION DAMAT(2,150),MDAT(100,DLAT(150),DAMAT(2,150)
DIMENSION AMASS(300),EVEC(100,10),LAMASS,10)
CALL FINISH( 'EQUATE 99=PRINT;' )

IUNIT DEFINES THE INPUT AND OUTPUT FILES :-

IUNIT=5 IS DATA SOURCE FILE
IUNIT=6 IS TEMPORARY STORAGE FOR INTERMEDIATE DATA
IUNIT=7 IS FINAL OUTPUT FILE
IUNIT=8 IS DAMAGE RATIO FILE ( SEPARATE FROM OTHER FINAL
OUTPUT FILE TO MAKE PLOTTING OF RESULTS EASIER)
IUNIT=7

CALL CTRL TO READ IN DATA OF STRUCTURE, TITLE AND PROGRAM
OPTIONS.

CALL CTRL(TITLE.NRJ,NRM,E,G,J,AMAX,ISPEC,DAMP,
1 INELAS,NNODES,NPRINT,HARD)

IDIM DIMENSIONS THE STIFFNESS MATRIX FOR SUBROUTINES

IDIM=8000

CALL SETUP TO READ AND TO ECHO PRINT MEMBER AND JOINT DATA
-HALF BANDWIDTH AND NUMBER OF UNKNOWNS ARE CALCULATED

CALL SETUPNRM,E,G,YM,VM,DM,ND,NP,AREA,CROHOM,DAMAT,
1 NRJ,AV,KL,KG,NU,NB,SDAMP,BMCP,IUNIT,EXTL,EXTG)

SET IFLAG EQUAL TO 1 IF ONLY ONE ITERATION IS REQUIRED
HERE IFLAG IS SET EQUAL TO 0

IFLAG=0

CHECK IF IDIM HAS BEEN ASSIGNED LARGE ENOUGH

LSTM = LENGTH OF ONE-DIMENSIONAL STIFFNESS MATRIX

LSTM=NU*NB

IUNIT=7,IUNIT=5 write(7,10) LSTM,IDIM

FORMAT(/// 'PROGRAM STOPPED'.//'LENGTH OF STIFFNESS MATRIX

10 IG:/PROVIDED STORAGE (IDIM)**1.)

IF (LSTM,IDIM) STOP

ICOUNT IS THE NUMBER OF TIMES MAIN MSSM SUBROUTINE IS CALLED
ICOUNT IS INITIALIZED TO ZERO HERE.

ICOUNT=0

CALL MASS TO READ AND ASSIGN MASSES TO NODES
- ASSEMBLE THE MASS MATRIX : AMASS

CALL MASS(NRJ,ND,AMASS,IUNIT,NRJ,NMASS,MDAT)

CHECK IF IDIM HAS BEEN SUFFICIENTLY DIMENSIONED

IUNIT=4)IUNIT=7

IUNIT=2)IUNIT=7

FORMAT(/// 'THE VALUE OF IDIM IS SMALLER THAN 'PRITZ' REQUIRES')

IUNIT=7

REASSIGN OUTPUT TO TEMPORARY FILE 6
C

UNIT=6
C
C IF ONLY ELASTIC ANALYSIS IS REQUIRED: RESET CONTROL FLAGS
C SET IFLAG=1 TO INDICATE ONLY ONE ITERATION IS REQUIRED
C
140 IF(INELAS.NE.0) GO TO 70
141 WRITE(7,110)
142 UNIT=7
143 IFLAG=1
144 WRITE(7,110)
145 CONTINUE
70

C
C SET THE MAXIMUM NUMBER OF ITERATIONS.
C
150 IMAX=1+
151 IF(INELAS.NE.0) IMAX=INELAS
152 IM=IMAX-1
153 C I * THE NUMBER OF ITERATIONS PERFORMED
154 C 1=0
155 C SET LOCK TO 0 FOR NORMAL CONVERGENCE PROCEDURE
156 C
157 C LOCK=0
158 C BETA IS A FACTOR USED IN SPEEDING CONVERGENCE (0 < BETA < 1).
159 C BETA = .0, EFFECTIVELY SHUTS OFF CONVERGENCE SPEEDING ROUTINE
160 C
161 C SET ERROR RATIO OF MOMENTS OF YIELDED MEMBERS (BMERR).
162 C A VALUE OF 0.05 HERE ENSURES YIELDED MEMBERS ARE WITHIN
163 C 5 PERCENT OF THEIR CAPACITY.
164 C
165 BMERR=0.05
166 C
167 C - SET CONVERGENCE LIMIT FOR CHANGE IN DAMAGE RATIO
168 C
169 C DAMERR = 0.01 ENSURES THAT THE MAXIMUM DAMAGE RATIO CHANGE
170 C IN THE FINAL ITERATION IS ONE PERCENT - FOR DAMAGE RATIOS
171 C ABOVE 5.0
172 C - THOSE DAMAGE RATIOS BELOW 5.0 WILL CONVERGE TO THEIR
173 C ABSOLUTE VALUE DIFFERENCE BEING TEN TIMES THE RATIO
174 C
175 C DAMERR=0.01
176 C
177 C - INITIALIZE ARRAY USED IN SPEEDING CONVERGENCE
178 C DO BO MEM=1,NRM
179 C DAM(1,MEM)=DAMAT(1,MEM)
180 C DAM(2,MEM)=DAMAT(2,MEM)
181 DO CONTINUE
182 C
183 C FINISHED INPUT OF DATA AND INITIAL ACTIVITIES.
184 C
185 C INCREMENT ITERATION COUNTER :-
186 C
187 C
188 C
189 C 100 I=I+1
190 WRITE(UNIT,110)
191
192 C
193 C
194 C
195 C
196 C
197 C
198 FORMAT(10D15.15)
199 WRITE(UNIT,120)
200 120 FORMAT(10D15.15)
201 C CALL BUILD TO COMPUTE THE MEMBER AND GLOBAL STIFFNESS MATRIX
202 C CALL BUILD(JN,JB,XM,YM,DM,AREA,CROMM,AV,E.G.DAMAT,KL,KG,
203 NRM,S,IDIM,EXL,EXTG)
204 C
205 CALL SHECK TO CHECK THE CONDITION OF THE STIFFNESS MATRIX
206 C
207 C CALL SCHECK(S,JN,JB,IDIM,SRATIO)
208 C
209 C CALL EIGEN TO COMPUTE THE FREQUENCIES AND MODE SHAPES FOR
210 C THE SUBSTITUTE STRUCTURE
211 C
212 C CALL EIGEN(JN,JB,S,IDAM,AMAS,VAL,EIG,EVODES,UNIT,ISPEC. 
213 1
214 C AMAX,(ICOUNT,MDOF,INELAS)
215 C
216 C INSERT HEADINGS FOR ITERATION PROGRESS (FOR INELASTIC
217 C ANALYSIS ONLY)
218 C
219 C IFINELAS.EQ.0.OR.(COUNTE.NE.0) GO TO 105
220 C
221 C WRITE(7,115)
222 115 FORMAT(10D15.15)
223 WRITE(7,110)
224 110 FORMAT(10D15.15)
225 WRITE(7,95) DAMERR
226 95 FORMAT(6F15.8)
227 WRITE(99,90)
228 90 FORMAT(70F15.8)
229
230 C AFTER 9 ITERATIONS BETA IS REASSIGNED FROM 0.0 TO 0.8
231 C IF NO. ABOVE CAPACITY = 0, SET BETA=0.0
232 C IF (INELAS.EQ.0) BETA=0.0
233 C IF (INELAS.EQ.0) BETA=0.0
234 C DVAR = THE LARGEST DAMAGE RATIO DIFFERENCE BETWEEN THIS
235 C AND THE LAST ITERATION
236 C
237 C DVAR=0.0
238 C
239 C CALL MOD3 - THE MAIN SUBROUTINE FOR THE WSSM
240 C
241 C CALL MOD3(ICOUNT,ISPEC,NRJ,NRM,NJ,NB,NMODES,S,IDIM,ND,NP,XM,YM,
242 C X2,1
243 C DRESP,AMAT,AM,AREA,CROMM,AV,E.G.DAMAT,KL,KG,
244 C 2
245 C DAMAT,EVODES,UNIT,ISPEC. 
246 C
247 C OUTPUT DAMAGE RATIOS ON UNIT 10
248 C
249 C 206 C OUTPUT DAMAGE RATIOS ON UNIT 10
250 C
251 C 208 C OUTPUT NUMBER OF MEMBER IN EXCESS OF CAPACITY AND LARGEST
252 C DIFFERENCE FROM PREVIOUS ITERATION DAMAGE RATIOS
253 C
254 C 209 C OUTPUT RATIO OF LARGEST TO SMALLEST NUMBER IN DIAGONAL
255 C STIFFNESS MATRIX (SRATIO)
256 C
257 C
258 C
259 C
260 C
261 C
262 C
263 C
CONTINUE

GO TO 100

MAXIMUM NUMBER OF ITERATIONS

IF(I_FLAG.EQ.1 AND I.GE.IMAX) GO TO 180

IF(I_FLAG.EQ.0 AND I.GE.IMAX) GO TO 200

IF(I.Flag.EQ.1 AND I.GE.IMAX) GO TO 220

IF(I.Flag.EQ.0 AND I.GE.IMAX) GO TO 240

IF(I.GE.IM) GO TO 250

IF(I.EQ.1 AND I.GE.IMAX) GO TO 260

IF(I.FLAG.EQ.1 AND I.GE.IMAX) GO TO 270

READ(I5,10) INELAS,NMODES,NPRINT,ISPEC,AMAX,DAMPIN

10 FORMAT(15I5)

READ(I5,10) INELAS,NMODES,NPRINT,ISPEC,AMAX,DAMPIN

10 FORMAT(15I5)

DAMPIN IS THE PROPORTION OF CRITICAL DAMPING USED IN ELASTIC

ANALYSIS OR THE FIRST ITERATION OF THE MSSM.

NPRINT IS A FLAG TO INDICATE IF ONLY ELASTIC ANALYSIS IS REQUIRED.

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SUBROUTINE SETUP(NRM, E, G, XM, YM, DM, ND, BP, AREA, CDMOM, DAMAT, JNL, JNG, BJ, BLCAP, JUNIT, EXTLIMBR, EXTG)

C SET UP THE FRAME DATA
C DIMENSION KL(NRM), KG(NRM), AREA(NRM), CDMOM(NRM), DAMAT(NRM), ND(K,J), BP(NRM), CI(NRM), NU(NJ, NB), SDAMP(NRM), BMEMAT(NRM)

C CONVERTED TO FOOT UNITS IN ROUTINE

C E AND G IN KSF
C K(J) AND Y(J) IN FEET
C MEMBER EXTENSIONS EXTG AND EXTL ARE IN FEET.
C AREAS IN SQ. INCHES. CDMOM(1) IN INCHES**4
C CONVERTED TO FOOT UNITS IN ROUTINE

C WRITE (IUNIT,230)
C WRITE (IUNIT,240)
C C READ IN JOINT DATA AND COMPUTE NO. OF DEGREES OF FREEDOM
C NU=1
C DD 50 I=1,NJ
C READ (5,250) JN, ND(I,1), ND(I,2), ND(I,3), DJ, YJ(I)
C DD K=1,3
C IF(DJ(I,J)>0.0 .AND. YJ>0.0) WRITE(7,60) I, J, DJ, YJ(I)
C NU=NU+1
C GO TO 40
C
C DO 20 K=1,3
C NU=NU+1
C END

C CONVERED TO KSI

C IF DAMAGE RATIOS ARE LESS THAN ONE SET EQUAL TO ONE
C DAMAT(1,1)=1.0
C DAMAT(2,2)=1.0
C C COMPUTE MEMBER LENGTH (DL)=LENGTH BETWEEN JOINTS-RIGID EXTENSIONS
C UL=UL(MBR)
C JL=JL(MBR)
C XM(MBR)=(XJL)-UL
C YM(MBR)=(YJL)-UL
C C INPUT EXTENSIONS TO MEMBER
C EXTLIMBR=EXTLIMBR+EXTG(MBR)
C XM=MXMRB+1.0-EXTLIMBR
C YM=MYMRB+1.0-EXTLIMBR
C C RESET NEGATIVE VALUES OF ZERO TO ZERO
C IF(XM<0.0) XM=0.0
C IF(YM<0.0) YM=0.0
C C IF NEGATIVE LENGTHS OF MEMBER
C (PROBABLY CAUSED BY INCORRECT USE OF MEMBER EXTENSIONS)
C IF(XM<0.0) WRITE(7,60) MBR
C IF(YM<0.0) WRITE(7,60) MBR
C C CHECK FOR NEGATIVE LENGTHS OF MEMBER
C IF(XM<0.0) WRITE(7,60) MBR
C IF(YM<0.0) WRITE(7,60) MBR
C C PROGRAM HALTED: ZERO OR <VE LENGTH FOR MEMBER",16)
C C CONTINUE
C C C YLEN=YM(MBR)
C C PRINT ERROR MESSAGE IF ATTEMPT TO HAVE RIGID EXTENSIONS
C C ON VERTICAL MEMBERS.
C IF(XM<0.0) WRITE(7,80) I, MBR
C 80 FORMAT(' ERROR-HAVE END EXTENSIONS ON NON-HORIZONTAL MEMBER',I4)
C C PRINT ERROR MESSAGE IF ATTEMPT TO HAVE RIGID EXTENSIONS ON
C A NON FIX-FIX TYPE MEMBER
C IF(YM<0.0) WRITE(7,90) MBR
C 90 FORMAT(' ERROR-HAVE RIGID EXTENSIONS ON MbayED MEMBER',I4)
SUBROUTINE BUILD(NL,XB,YN,DM,NP,AREA,CRMM,AV,E,G,DAMRAT,1)

! THIS SUBROUTINE WORKS IN DOUBLE PRECISION
! THIS SUBROUTINE CALCULATES THE STIFFNESS MATRIX OF EACH
MEMBER AND ADDS IT INTO THE STRUCTURE STIFFNESS MATRIX.
! THE FINAL STIFFNESS MATRIX'S IS RETURNED.
! THIS SUBROUTINE IS SIMILAR TO ONE THAT WOULD BE USED IN
NORMAL FRAME ANALYSIS.
! DIFFERENCES INCLUDE USING CRACKED MOMENT OF INERTIA INSTEAD
OF THE GROSS SECTION.
! DAMAGE RATIOS ARE USED AND FLEXURAL
! STIFFNESSES MODIFIED ACCORDING TO THESE RATIOS.
! IDIM IS THE DIMENSIONING SIZE OF THE STRUCTURE STIFFNESS MATRIX.
!
! REAL*8 SM(2I),S(DIM)
DIMENSION XM(NM),YM(NM),DM(NM),NP(E,NM),AREA(NM),1
! CRMM(NM),AV(NM),DAMRAT(2,NM),KL(NM),KG(NM)
DIMENSION E(TNM),G(TNM)
!
REASSIGN YOUNG'S MODULUS TO DOUBLE PRECISION VARIABLE EMD
! MODIFIED DOUBLE
BEGIN MEMBER LOOP
DO 10 1=1,NM
!
C GIVE MEMBERS INITIAL ELASTIC DAMPING
SDAMP(MBR)=0.02
C ASSIGN MEMBER DEGREES OF FREEDOM
NP1(MBR)+NP(D2,MBR)=1,4
NP2(MBR)+NP(D2,MBR)=1,2
NP3(MBR)+NP(3,MBR)=1,3
NP4(MBR)+NP(4,MBR)=1,1
NP5(MBR)+NP(D5,MBR)=1,2
NP6(MBR)+NP(D6,MBR)=1,3
C DETERMINE THE HIGHEST DEGREE OF FREEDOM FOR EACH MEMBER STORING
C THE RESULT IN 'MAX'
DO 120 K=1,6
IF(NP(K,MBR)=MAX) 110,110,100
MAX=NP(K,MBR)
110 CONTINUE
120 CONTINUE
C DETERMINE THE MINIMUM DEGREE OF FREEDOM FOR EACH MEMBER, NOTE THAT
C FOR STRUCTURES WITH GREATER THAN 330 JOINTS INITIAL VALUE OF MIN
C WILL HAVE TO BE INCREASED FROM ITS PRESENT POINT OF 1000.
MIN=1000
DO 140 I=1,IDIM
REAL*8 RF,GMOO,CMOM,DRAT,F,H
140 CONTINUE
WRITE (UNIT,310) MBR,ULNL(MBR),JNG(MBR),EXTL(MBR),DM(MBR),1
WRITE (UNIT,320) MBR,AREA(MBR),CPMOM,AV(MBR),BMCAP(MBR),KL(MBR),
Kg(MBR)
WRITE (UNIT,330) MBR,AREA(MBR),CPMOM,AV(MBR),BMCAP(MBR),KL(MBR),
Kg(MBR)
WRITE (UNIT,310) MBR,ULNL(MBR),JNG(MBR),EXTL(MBR),DM(MBR),1
!
DO 360 K=1,6
IF(NP(K,MBR)) 150,150,130
150 CONTINUE
!
MEMBER AND ADDS IT INTO THE STRUCTURE STIFFNESS MATRIX.
NORMAL FRAME ANALYSIS.
!
FORMAT(13,14,2F10.3,2X,15E12.1)
FORMAT(14,2F10.3,2X,15E12.1)
RETURN
C ZERO MEMBER STIFFNESS MATRIX

C DO 20 J=1,21
C SM(J)=1.0000
C 20 CONTINUE

C ASSIGN MEMBER PROPERTIES TO DOUBLE PRECISION VARIABLES
C LONE=DOUBLE(EXTL(1))
C LTWO=DOUBLE(EXTL(2))
C YM1=DOUBLE(YM1(1))
C XMI=DOUBLE(XM1(1))
C OM1=DOUBLE(OM1(1))
C YM2=DOUBLE(YM2(1))
C XM2=DOUBLE(XM2(1))
C CMOM1=DOUBLE(CMOM1(1))
C AVI=DOUBLE(AVI(1))

C OBTAIN EFFECTIVE DAMAGE RATIO FROM 'DAMCAL'
C CALL DAMCAL(DAMMAT,DRAT1(1))

C CALL DAMCAL(DAMMAT,DRAT1(1))
C DM2=DM1*DM1
C XM2=XM1*XM1
C YM2=YM1*YM1
C XM1=XM2/DM2
C YM1=YM2/DM2
C F=AREA*(DM1*DM2)
C H=0.0000
C SHEAR DEFLECTIONS ARE IGNORED WHENEVER G OR AV IS ZERO.
C IFAV(1)>E:0.00 OR G>0.0, GO TO 30
C H=12.0000*MOD(CMOM1/(AV1*GMOD*DM2)
C 30 XM2F=XM2+F
C YM2F=YM2+F
C XM2F=XM2+F
C YM2F=YM2+F
C HMOM=XM2F+YM2F
C SM(17)=XM1*HMOM
C SM(10)=-XM1*HMOM
C SM(7)=XM1*HMOM
C SM(3)=SM(3)-HMOM
C SM(6)=SM(6)-HMOM
C SM(14)=SM(14)-HMOM
C SM(9)=SM(9)-HMOM
C SM(15)=SM(15)-HMOM
C SM(18)=SM(18)-HMOM
C SM(20)=SM(20)-HMOM
C SM(21)=SM(21)-HMOM
C ADD IN TERMS FOR RIGID END EXTENSIONS.
C SM(3)+SM(3)-LONEY
C SM(6)+SM(6)-LLOWD
C SM(9)+SM(9)-LLOWD
C SM(12)+SM(12)-LLOWD
C SM(15)+SM(15)-LLOWD
C SM(18)+SM(18)-LLOWD
C SM(21)+SM(21)-LLOWD
C ADD IN TERMS WHICH ARE COMMON TO PIN-FIX,FIX-PIN, AND
C FIX-FIX MEMBERS
C LONE=LONE+XM1*RF*DRAT1(4)
C LFIXD=LONE+XM1*RF*DRAT1(4)
C LLOWD=LONE+XM1*RF*DRAT1(5)
C LLOWD=LONE+XM1*RF*DRAT1(5)
C 60
DO 180 J1=1,J-1+12-J/2
180 CONTINUE

DO 180 L=J-6
180 CONTINUE

K=(NP(J,1)-1)*NB1+NP(J,1)
140 K=INP(L,1)-1)'NB1+NP(J,1)
150 K=(NP<J,1)-1  )'NB1+NP(J,1)
160 K=INP(J,1)-1)'NB1+NP(J,1)
170 N-J=HL
180 CONTINUE

SUBROUTINE DAMCAL(DAMRAT,ORATN,NOM)
DIMENSION OAMRAI(2,NOM)
REAL DRAT 1(6 I .DBLE
DRATI(1 ) =DBLE (UAMRAK 1 .NOM) )
DRATI(2) = DBLE(DAMRAT(2.NOM))
DRATI(3) = 1 . D 0/( .09500+.2D0*(DRAT I( 1 ) + DRATI(2))+.50500*DRATI( 1)
1 •DRATU2I)
DRATI( 1 ) = 1 D0/DRAT1( 1)
DRAT 1(2) =1.DO/DRAT 1(2)
DRATI(4) = ( 2 DO'DRAT I( 1 )+ DRATI(3)I/3.00
OR 1T1 ( 2 l+DRATI ( 3 ) )/3 .00
DRATI(6) = (DRAT II 1 )+DRATI(2)+DRATI(3))/3.DO
RETURN
END

SUBROUTINE MASSNU.ND.AMASS.IUNIT,NRJ,NMASS,MDOF)
DIMENSION ND(3.NRJ). MOOF(IOO), AMASS(NU)
READ IN NO. OF NODES WITH MASS
DO 10 I=1.NU
AMASS!I)=0.
10 CONTINUE
READ IN X-MASS.V-MASS AND ROT. MASS (IN UNITS OF WEIGHT)
DO 50 1=1,NMASS
READ (5.140) JN. WTX, WTV, WTR
WRITE (IUNIT.150) JN, WTX, WTV, WTR
N1=ND(1,JN)
N2=ND(2.JN)
N3=ND(3.JN)
IF(N1.F.0.O) GO TD 20
AMASS!N1)=AMASS(N1)+(WTX/32.2>
20 IFIN2.F.0.OI GO TO 30
AMASS!N2l=AMASS(N2)+(WTY/32.2>
3n IF IN3.EO.O) GO TO 40
AMASS!N3)=AMASS(N3) + (WTR/32.2 I
•10 CONTINUE
50 CONTINUE
OUTPUT THE DEGREES OF FREEOOM WITH MASS AND ASSIGNED MASS.
JCNT=1
WRITE!IUNIT,70)
DO 60 IDOF=1,NU
RMASS = AMASS( IDOF)
IF(RMASS.EO.0.0) GO TO 60
MDOF(JCNT)=IDOF
WRITE!IUNIT.80) JCNT.MDOF(JCNT).RMASS
JCNT=JCNT+1
60 CONTINUE

READ (5,140) JN, WTX, WTY, WTR
WRITE (IUNIT.150) JN, WTX, WTY, WTR
N1=ND(1, JN)
N2=ND(2, JN)
N3=ND(3, JN)
IF(N1.EO.O) GO TO 20
AMASS(N1)=AMASS(N1)+(WTX/32.2)
20 IF IN2.EO.O) GO TO 30
AMASS(N2)=AMASS(N2)+(WTY/32.2)
3n IF IN3.EO.O) GO TO 40
AMASS(N3)=AMASS(N3)+(WTR/32.2)
•10 CONTINUE

READ (5,140) JN, WTX, WTY, WTR
WRITE (IUNIT.150) JN, WTX, WTY, WTR
N1=ND(1, JN)
N2=ND(2, JN)
N3=ND(3, JN)
IF(N1.EO.O) GO TO 20
AMASS(N1)=AMASS(N1)+(WTX/32.2)
20 IF IN2.EO.O) GO TO 30
AMASS(N2)=AMASS(N2)+(WTY/32.2)
3n IF IN3.EO.O) GO TO 40
AMASS(N3)=AMASS(N3)+(WTR/32.2)
•10 CONTINUE

DO 50 I=1.NU
AMASS(I)=0.
50 CONTINUE
READ IN X-MASS,Y-MASS AND ROT. MASS (IN UNITS OF WEIGHT)
DO 50 I=1.NU
AMASS(I)=AMASS(I)+1.WX/32.2
50 CONTINUE
WRITE (IUNIT.100)
WRITE (IUNIT.110) NMASS
WRITE (IUNIT.120)
WRITE (IUNIT.130)
RETURN

This subroutine sets up the mass matrix.

Subroutine DAMCAL(DAMRAT,ORATN,NOM)
Effectivf damage ratio calculation.

Dimension ND(J, I )=Degrees of freedom of J th joint.
AMASS(I)=Mass matrix. I is the degree of freedom of applied mass.
Masses are lumped at nodes. The mass matrix is diagonalized.
Dimension ND(J, I )=Degrees of freedom of J th joint.
AMASS(I)=Mass matrix. I is the degree of freedom of applied mass.
Masses are lumped at nodes. The mass matrix is diagonalized.

This subroutine sets up the mass matrix.
**SUBROUTINE EIGEN**

**DESCRIPTION**

This subroutine computes a specified number of natural frequencies and associated mode shapes.

**PARAMETERS**

- **NU**: Number of degrees of freedom.
- **NB**: Half bandwidth.
- **S**: Stiffness matrix stored by columns.
- **IDIM**: Dimension of matrices.
- **AMASS**: Mass matrix.
- **EVAL**: Eigenvalues.
- **EVEC**: Eigenvectors (mode shapes).
- **NMODES**: Number of modes to be computed.
- **IUNIT**: Unit number for printing.
- **ICOUNT**: Count of cycles.
- **INELAS**: Integer for elastic period.

**FUNCTIONS**

1. Computes the number of nonzero mass matrix entries.
2. Computes the number of nonzero stiffness matrix entries.
3. Converts matrices to single precision.
4. Converts eigenvectors to include only degrees of freedom.
5. Converts members of EVAL from omega squared to omega.
6. Computes frequencies and periods.
7. Computes frequencies and periods with mass assigned to them.
8. Prints eigenvalues and eigenvectors.
9. Converts eigenvalues and eigenvectors (mode shapes).
10. Skips printing intermediate data after several cycles.
11. Outputs initial elastic period.
12. Outputs natural frequencies.

**METHODS**

- **DO Loop**: Iterates over the number of modes.
- **WRITE**: Outputs data to the unit.
- **CONTINUE**: Continues to the next iteration.

**EXAMPLE**

```plaintext
DO 10 JUICE=1,NMODES
    EVAL(JUICE)=SQRT(EVAL1)
    WRITE (IUNIT,210) NMODES, EVAL(JUICE)
END DO
WRITE (IUNIT,220) NMODES
```

**NOTES**

- **CONVERT MATRICES TO SINGLE PRECISION**
- **PRINT EIGENVALUES AND EIGENVECTORS (MODE SHAPES)**
- **EIGENVALUES (EVAL) ARE THE VALUES OF OMEGA SQUARED.**
- **SKIP PRINTING INTERMEDIATE DATA AFTER SEVERAL CYCLES.**
- **IF(ICOUNT GT 3 AND IUNIT EQ 6) GO TO 70**
- **WRITE (IUNIT, 170)**
- **WRITE (IUNIT, 180)**
- **WRITE (IUNIT, 190)**
- **WRITE (IUNIT, 200)**
- **WRITE (IUNIT, 210)**
- **WRITE (IUNIT, 220)**

**REFERENCES**

- **EIGENVALUES AND EIGENVECTORS**
- **MATRICES TO SINGLE PRECISION**
- **EIGENVECTORS**
- **EIGENVALUES**
- **EIGENVECTORS TO ONLY INCLUDE DEGREES OF FREEDOM**
- **EIGENVECTORS WITH MASS ASSIGNED TO THEM**
- **EIGENVECTORS FOR INITIAL ELASTIC PERIOD**
- **EIGENVECTORS FOR NATURAL FREQUENCIES**
SUBROUTINE MOD3(ICOUNT, ISPEC, NRM, NR, NMOD, SDAMP, DAMRAT, KL, KG, SDAMP, BCMAP, AMASS, EVEC, EVAL, AMAX, 1SIGN, IUNIT, BMERR, IFLAG, EXTL, EXTG, BETAM, DAMB, VARY, INELAS, DAM IN, NPRINT, HARO, 0L01N, 0LDSA, LOCK)

C SUBSTITUTE STRUCTURE METHOD FOR RETROFIT
C THIS SUBROUTINE COMPUTES JOINT DISPLACEMENTS AND MEMBER FORCES FOR MODIFIED STRUCTURE. NEW DAMAGE RATIOS WILL BE CALCULATED AND RETURNED.
C
REAL*8 S(IDIM)
DIMENSION ND(3,NR), NP(6,NR), XM(NR), YM(NR), OM(NR), AREA(NR), CRMOM(NR), DAMRAT(2,NR), KLIN(NR), KG(NR), EVEC(NR, NMODE), EVAL(NMODE), SDAMP(NR), AV(NR), AMASS(NR)
DIMENSION BMASS(100), IDOFUOO, ALPHA (20), RMS(7,150), F(300), EXTL(NR), EXTG(NR), BCMAP(NR), DAMB(2,NR), BETAM(NODE), OLDN(1), OLDSA(1)

REAL'S DRATIO, DET
C
C CALCULATE THE MODAL PARTICIPATION FACTOR :
C JJ = TEMPORARY VARIABLE USED IN THE FOLLOWING LOOP ONLY
JJ=1
C
DO 30 JDOF=1.NR
IF(AMASS(JDOF).EQ.0.) GO TO 30
BMASS(J)=AMASS(JDOF)
IDOF(J)=JDOF
JJ=JJ+1
30 CONTINUE
C
MCOUNT=JJ-1
C
0070 MODE=1,NMODES
AMT=0.
AMB=0.
C
DO 70 J=1,MOUNT
AMT=AMT+BMASS(J)*EVEC(J,MODE)
AMB=AMB+BMASS(J)*(EVEC(J,MODE)+2)
70 CONTINUE
C
C WHEN KK=1, MODAL FORCES FOR UNDAMPED SUBSTITUTE STRUCTURE ARE COMPUTED. THEY ARE USED TO COMPUTE 'SMEARED' DAMPING VALUES.
C WHICH ARE USED TO CALCULATE THE ACTUAL RESPONSE OF THE SUBSTITUTE STRUCTURE
C
INDEX=1

C DO 800 KK=1,2
C SET PRINT FLAG FOR MODAL OUTPUT (0=OFF)
INTPR=1
IF(KK.EQ.1) INTPR=0
IF(IFLAG.EQ.O.OR.NPRINT.EQ.O) INTPR=0
IF (ICOUNT .NE .0 ) GO TO 100
C
C SET DAMPING RATIOS TO 'APPROPRIATE' VALUES FOR INITIAL TRIAL.
C DO 790 MODE=1,NMODES
BETAM(MODE)=DAMRAT(MODE)
790 CONTINUE
C
ICOUNT=ICOUNT+1
C OUTPUT THE SMEARED DAMPING RATIOS (FOR DAMPED CASES)
IF(IUNIT.EQ.6.AND.ICOUNT.GT.25) GO TO 150
IF(KK.EQ.1) GO TO 150
WRITE(IUNIT,130) MODE, BETAM(MODE)
130 FORMAT('MODE',2X,'SMEARED DAMPING RATIO'),
140 FORMAT(' ',IX,I3.7X.F10.5)
150 CONTINUE
C
C CALCULATE THE MODAL DISPLACEMENT VECTOR
C
DO 570 MODE=1,NMODES
TN=6.28318531/(EVAl(MODE))
WN=EVAL(MODE)
570 CONTINUE
C
OUTPUT THE SMEARED DAMPING RATIOS FOR DAMPED CASES
IF(IUNIT.EQ.6.AND.ICOUNT.GT.25) GO TO 150
IF(KK.EQ.1) GO TO 150
WRITE(IUNIT,130) MODE, BETAM(MODE)
130 FORMAT('MODE',2X,'SMEARED DAMPING RATIO')
140 FORMAT(' ',IX,'F10.5')
150 CONTINUE
C
C CALCULATE THE MODAL DISPLACEMENT VECTOR
C
DO 70 J=1,NMODES
AMD=AMD+BMASS(J)*(EVEC(J,MODE)+2)
70 CONTINUE
C
C CALCULATE THE MODAL DISPLACEMENT VECTOR
C
DO 1000 MD=1,150
RMS(J,MD)=0.
1000 CONTINUE
C
C WHEN KK=1, MODAL FORCES FOR UNDAMPED SUBSTITUTE STRUCTURE ARE COMPUTED. THEY ARE USED TO COMPUTE 'SMEARED' DAMPING VALUES.
C WHICH ARE USED TO CALCULATE THE ACTUAL RESPONSE OF THE SUBSTITUTE STRUCTURE
C
INDEX=1

C DO 800 KK=1,2
C SET PRINT FLAG FOR MODAL OUTPUT (0=OFF)
INTPR=1
IF(KK.EQ.1) INTPR=0
IF(IFLAG.EQ.O.OR.NPRINT.EQ.O) INTPR=0
IF (ICOUNT .NE .0 ) GO TO 100
C
C SET DAMPING RATIOS TO 'APPROPRIATE' VALUES FOR INITIAL TRIAL.
C DO 790 MODE=1,NMODES
BETAM(MODE)=DAMRAT(MODE)
790 CONTINUE
C
ICOUNT=ICOUNT+1
C OUTPUT THE SMEARED DAMPING RATIOS (FOR DAMPED CASES)
IF(IUNIT.EQ.6.AND.ICOUNT.GT.25) GO TO 150
IF(KK.EQ.1) GO TO 150
WRITE(IUNIT,130) MODE, BETAM(MODE)
130 FORMAT('MODE',2X,'SMEARED DAMPING RATIO')
140 FORMAT(' ',IX,'F10.5')
150 CONTINUE
C
C CALCULATE THE MODAL DISPLACEMENT VECTOR
C
DO 570 MODE=1,NMODES
TN=6.28318531/(EVAl(MODE))
WN=EVAL(MODE)
570 CONTINUE
C
OUTPUT THE SMEARED DAMPING RATIOS FOR DAMPED CASES
IF(IUNIT.EQ.6.AND.ICOUNT.GT.25) GO TO 150
IF(KK.EQ.1) GO TO 150
WRITE(IUNIT,130) MODE, BETAM(MODE)
130 FORMAT('MODE',2X,'SMEARED DAMPING RATIO')
140 FORMAT(' ',IX,'F10.5')
150 CONTINUE
C
C CALCULATE THE MODAL DISPLACEMENT VECTOR
C
DO 70 J=1,NMODES
AMD=AMD+BMASS(J)*(EVEC(J,MODE)+2)
70 CONTINUE

DAMP=BETA(MODEN)

IF (MODEN.NE.1 .OR. LOCK.EQ.0)
   CALL SPECI(1,SPEC,DAMP,TN,AMAX,SA,WN,SBND,SVBND,SOBND)

IF (MODEN.EQ.1)
   CALL STACK(0,OLDSA,SA,OLDIN,TN,ISPEC,LOCK,ICOUNT,IFLAG,
   21  IUNIT,AMAX,DAMP,K)

IF (MODEN.EQ.1 AND KK.EQ.2) WRITE(99,205) TN,SA
IF (MODEN.EQ.1 AND KK.EQ.2 AND IFLAG.NE.1) WRITE(7,205) TN,SA
FORMAT (50X,'PERIOD ',',F6.3,'), ('SA ',',F6.3',')

205 CONTINUE

206 C LIST MEMBER FORCES IF DOING ELASTIC ANALYSIS ONLY

207 C IF(INFR.EQ.0) GO TO 180

208 IF(INPRINT.LT.MODEN) GO TO 180

209 IF(INTPR.EQ.O) GO TO 180

210 C WRITE MODAL CONTRIBUTION FACTOR

211 C CHECK IF MODAL PARTICIPATION FACTOR IS ZERO

212 C IF ALPHA IS ZERO MODAL FORCES AND DISPLACEMENTS WILL BE ZERO

213 C IF(ALPHA(MODEN).NE.0) GO TO 200

214 WRITE(IUNIT,190)

215 FORMAT(/' MODAL PARTICIPATION ,FORCES AND DISPL. =ZERO:')

216 CONTINUE

217 C ZERO LOAD VECTOR

218 DO 220 J-1,NU

219 FI(J)=0.

220 CONTINUE

221 C ZERO LOAD VECTOR

222 C COMPUTE LOAD VECTOR

223 C IF(0.EQ.0) GO TO 200

224 WRITE(IUNIT,90)

225 FORMAT(/' LOAD VECTOR

226 CONTINUE

227 C IF ALPHA IS ZERO MODAL FORCES AND DISPLACEMENTS WILL BE ZERO

228 C IF(ALPHA(MODEN).NE.0) GO TO 200

229 WRITE(IUNIT,190)

230 FORMAT(/' MODAL PARTICIPATION ,FORCES AND DISPL. =ZERO:')

231 CONTINUE

232 C COMPUTE LOAD VECTOR

233 C IF(0.EQ.0) GO TO 200

234 WRITE(IUNIT,90)

235 FORMAT(/' LOAD VECTOR

236 CONTINUE

237 C NOTE THAT AS THESE FORCES ARE BEING GENERATED FROM A

238 C LATERAL EXCITATION SPECTRUM THAT ONLY 'X MASSES' SHOULD

239 C BE USED. IN OTHER WORDS LATERAL ACCELERATION SHOULD NOT

240 C CAUSE NON HORIZONTAL INERTIA FORCES DIRECTLY.

241 C

242 C FF=0.

243 DO 220 J-1,NUCOUNT

244 II=100D(J)

245 FORMAT(1'E6.1',4E11.7)

246 FF=FF+II

247 CONTINUE

248 C DIGIT TOLERANCE

249 C

250 IF(N2.EQ.0) GO TO 270

251 RMS(1,JNT)=RMS(1,JNT)+DX**2

252 CONTINUE

253 IF(N2.EQ.0) GO TO 270

254 D**=D**+RMS(1,JNT)**2

255 CONTINUE

256 IF(N2.EQ.0) GO TO 270

257 D**=D**+RMS(1,JNT)**2

258 CONTINUE

259 IF(N2.EQ.0) GO TO 270

260 D**=D**+RMS(1,JNT)**2

261 CONTINUE

262 IF(N2.EQ.0) GO TO 270

263 RMS(2,JNT)=RMS(2,JNT)*DY**2

264 RMS(2,JNT)=RMS(2,JNT)*DY**2

265 RMS(2,JNT)=RMS(2,JNT)*DY**2

266 CONTINUE

267 IF(N2.EQ.0) GO TO 270

268 RMS(2,JNT)=RMS(2,JNT)*DY**2

269 RMS(2,JNT)=RMS(2,JNT)*DY**2

270 CONTINUE

271 C IF N2.EQ.0 GO TO 270

272 D**=D**+RMS(2,JNT)**2

273 RMS(2,JNT)=RMS(2,JNT)*DY**2

274 RMS(2,JNT)=RMS(2,JNT)*DY**2

275 CONTINUE

276 IF(N2.EQ.0) GO TO 270

277 RMS(2,JNT)=RMS(2,JNT)*DY**2

278 RMS(2,JNT)=RMS(2,JNT)*DY**2

279 RMS(2,JNT)=RMS(2,JNT)*DY**2

280 CONTINUE

281 IF(N2.EQ.0) GO TO 270

282 RMS(2,JNT)=RMS(2,JNT)*DY**2

283 RMS(2,JNT)=RMS(2,JNT)*DY**2

284 RMS(2,JNT)=RMS(2,JNT)*DY**2

285 CONTINUE

286 IF(N2.EQ.0) GO TO 270

287 RMS(2,JNT)=RMS(2,JNT)*DY**2

288 RMS(2,JNT)=RMS(2,JNT)*DY**2

289 RMS(2,JNT)=RMS(2,JNT)*DY**2

290 CONTINUE

291 C COMPUTE AND WRITE MODAL CONTRIBUTION FACTOR

292 C IF(N2.EQ.0) GO TO 270

293 RMS(2,JNT)=RMS(2,JNT)*DY**2

294 RMS(2,JNT)=RMS(2,JNT)*DY**2

295 RMS(2,JNT)=RMS(2,JNT)*DY**2

296 CONTINUE

297 IF(N2.EQ.0) GO TO 270

298 RMS(2,JNT)=RMS(2,JNT)*DY**2

299 RMS(2,JNT)=RMS(2,JNT)*DY**2

300 RMS(2,JNT)=RMS(2,JNT)*DY**2

301 CONTINUE

302 C CALL FORCE TO CALCULATE MEMBER FORCES AND SMOOTHED

303 C DAMPING RATIOS

304 CALL FORCE (NMM,KM,YM,DV,AV,NP,F,EXTL,EXTG,
   1 AREA,E,G,NPRINT,COMMDAMRAT,INPR,
   2 KL,KG,KK,SADM,NMODES,IUNIT,IFLAG,
   3 NMODES,IUNIT,RMS,BETAM)

305 C COMPUTE AND WRITE MODAL CONTRIBUTION FACTOR

306 CONWOD=SA*ALPHA(MODEN)

307 WRITE(IUNIT,550) MODEN,CONWOD

308 FORMAT(1'E6.1',4E11.7)

309 OUTPUT SPECTRAL ACCELERATION.

310 C IF(N2.EQ.0 OR MODEN.GT.NPRINT) GO TO 570

311 WRITE(IUNIT,560) DAMP,TN,SA

312 FORMAT(1'E6.1',4E11.7)

313 CONTINUE

314 OUTPUT SPECTRAL ACCELERATION.

315 C IF(N2.EQ.0 OR MODEN.GT.NPRINT) GO TO 570

316 WRITE(IUNIT,560) DAMP,TN,SA

317 FORMAT(1'E6.1',4E11.7)

318 CONTINUE

319 CALL PRINT RMS DISPLACEMENTS AND FORCES

320 C IF(N2.EQ.0 OR MODEN.GT.NPRINT) GO TO 570

321 WRITE(IUNIT,8401)
**Note: This code appears to be written in Fortran, a computer programming language.**

The code seems to be part of a program that calculates various forces and moments related to structural analysis, specifically focusing on modal analysis and damage calculation.

Here are some key points from the code:

- The program includes routines for outputting force and displacement information.
- It calculates modal participation factors and participation modes.
- There are sections for handling input data and various calculations involving forces and moments.
- The code also includes routines for handling modal force and moment calculations.

Overall, the code appears to be part of a larger program designed for structural analysis, possibly for civil engineering or similar fields.
IF (AVI(0,0,0).EQ.0.0) GO TO 90
GFAC = 12.0*EIS/(AVI*QDL*DL)*DRAT16
CONTINUE
90

C ASSIGN DISPLACEMENTS TO THEIR RESPECTIVE MEMBER DEGREES OF
C FREEDOM CHECK FOR PIN-PIN MEMBERS.

IF (K(I), EQ.0.0 AND K(I),1).EQ.0.0) GO TO 120
DEL = D(DEL(1)-DEL(2))*X1*(DEL(1)-DEL(4))/Y1/DL

120

BML = (2.0*EIS1/(1.0+GFAC))**15.0*DEL(4)*DRAT15

121

2*DRAT1 DEL(4)*DRAT15

122

BMG = (2.0*EIS1/(1.0+GFAC))**15.0*DEL(5)/DL

123

2*DRAT1 DEL(5)*DRAT15

124

SHEAR = (6.0*EIS1/(1.0+GFAC))**15.0*DEL(4)/DL

125

0.5BMG*BMG*DRAT11 + BMG*BMG*DRAT11

126

END
**CONVERGENCE SPEEDING ROUTINE FOLLOWS.**

IF (DIF1.GT.0.005.AND.DIF2.LT.-0.005) GO TO 40
IF (DIF1.LT.-0.005.AND.DIF2.GT.0.005) GO TO 40
RETURN

**COMPUTE DAMPING VALUE FOR THE MEMBER**

END

**SET OLDSA(1) = UPPER BOUND, OLDSA(I) = LOWER BOUND**

RETURN

**INSTABILITY DETECTED : START BINARY SEARCH ROUTINE**

C 14
C

1676 C IF (OLDSA(2).LT.OLDSA(1)) GO TO 52
1678 C TEMP=OLDSA(1)
1679 C OLDSA(1)=OLDSA(2)
1680 C OLDSA(2)=TEMP
1681 52 WRITE (99,55) OLDSA(1),OLDSA(2)
1682 C WRITE (7,55) OLDSA(1),OLDSA(2)
1683 C WRITE (99,120)
1684 C FORMAT (7,120)
1685 55 FORMAT (1/3X,'UPPER BOUND SA = ',F7.5,'X',LOWER BOUND SA = ',F7.5)
1686 60 OLDSA(1)=OLDSA(2))/2.
1687 C C CALCULATE SA AND CHECK FOR CONVERGENCE (IFLAG=1)
1688 C
1689 70 SA=OLDSA(3)*8./((6.+100.*DAMP)
1690 80 IF (IFLAG.EQ.1.AND.KK.EQ.2) GO TO 80
1691 RETURN
1692 C CHECK FOR REAL CONVERGENCE
1693 C
1694 90 CALL SPECTRI(SPEC,DAMP,TN,AMAX,SA,WN,SBND,SVBND,SOBND)
1695 100 IF (SADIF.LE.0.015) GO TO 100
1696 GO TO 90
1697 C CONVERGENCE LIMIT FOR SA IS 0.015
1698 C
1699 110 WRITE(99,110)
1700 110 FORMAT (1/,' ',12.55('-').'SADIF = .F6.4)
1701 C
1702 110 C SPECTRUM A
1703 110 IF (TN.LT.0.15) SA=25.*AMAX*TN
1704 110 IF (TN.GE.0.15 AND. TN.LT.0.4) SA=3.75*AMAX
1705 110 IF (TN.GE.0.4) SA=1.5*AMAX/TN
1706 110 GO TO 90
1707 C SPECTRUM B
1708 120 CONTINUE
1709 120 IF (TN.LT.0.1875) GO TO 20
1710 120 IF (TN.LT.0.53333333) GO TO 30
1711 120 IF (TN.LT.1.66666667) GO TO 40
1712 120 IF (TN.LT.1.81666667) GO TO 50
1713 120 SA=2.*AMAX/(TN-0.75)
1714 120 GO TO 90
1715 120 C SPECTRUM C
1716 120 CONTINUE
1717 120 IF (TN.LT.0.15) GO TO 20
1718 120 IF (TN.LT.0.38333333) GO TO 30
1719 120 IF (TN.LT.0.75) GO TO 50
1720 120 SA=0.5*AMAX/(TN-0.25)
1721 120 GO TO 90
1722 120 C NBC SPECTRUM
1723 120 CONTINUE
1724 120 IF (TN.LT.0.15) GO TO 20
1725 120 IF (TN.LT.0.38333333) GO TO 30
1726 120 IF (TN.LT.0.75) GO TO 50
1727 120 SA=40.0*AMAX
1728 120 SD=32.0*AMAX
1729 120 SACC=10.0*AMAX
1730 120 C  PRINT OUT A CAUTION NOTE SHOULD DAMPING BE LESS THAN 0.5%
1731 120 IF (DAMP .LT .0.050) WRITE(7,110)
1732 110 FORMAT ('CAUTION-DAMPING LESS THAN 0.5%')
1733 110 C SUBROUTINE SPECTRI(SPEC,DAMP,TN,AMAX,SA,WN,SBND,SVBND,SOBND)
1734 120 C COMPUTE MULTIPLICATION FACTOR FOR ACCELERATION AT DESIRED DAMPING

C SUBROUTINE SPECTRI(SPEC,DAMP,TN,AMAX,SA,WN,SBND,SVBND,SOBND)

C
<table>
<thead>
<tr>
<th>Line</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1804</td>
<td>IF (DAMP LE 0.02) AML = 4.2*(10.02-DAMP)/0.015 + 1.6</td>
</tr>
<tr>
<td>1805</td>
<td>IF (DAMP GT 0.02 AND DAMP LE 0.05*AML + 3 + (1.05-DAMP)/0.03) + 1.2</td>
</tr>
<tr>
<td>1806</td>
<td>IF (DAMP GT 0.05 AND DAMP LE 0.11<em>AML + 2.2</em>(10.1-DAMP)/0.05) + 0.8</td>
</tr>
<tr>
<td>1807</td>
<td>IF (DAMP GT 0.10<em>AML + 1.0</em>(11.00-DAMP)/0.015) + 1.2</td>
</tr>
<tr>
<td>1808</td>
<td>C</td>
</tr>
<tr>
<td>1809</td>
<td>C COMPUTE MULTIPLICATION FACTOR FOR VELOCITY AT DESIRED DAMPING.</td>
</tr>
<tr>
<td>1810</td>
<td>IF (DAMP GT 0.02) VML = 2.9*(10.02-DAMP)/0.015 + 0.5</td>
</tr>
<tr>
<td>1811</td>
<td>IF (DAMP GT 0.05) VML = 2.7 + (1.05-DAMP)/0.03 + 0.2</td>
</tr>
<tr>
<td>1812</td>
<td>C</td>
</tr>
<tr>
<td>1813</td>
<td>C COMPUTE MULTIPLICATION FACTOR FOR DISPLACEMENT AT DESIRED DAMPING.</td>
</tr>
<tr>
<td>1814</td>
<td>IF (DAMP GT 0.02) DML = 2.5 + ((0.05-DAMP)/0.03) + 0.2</td>
</tr>
<tr>
<td>1815</td>
<td>IF (DAMP GT 0.05) DML = 3.0 + ((0.05-DAMP)/0.03) + 1.2</td>
</tr>
<tr>
<td>1816</td>
<td>C</td>
</tr>
<tr>
<td>1817</td>
<td>C WRITE (99,150) TN</td>
</tr>
<tr>
<td>1818</td>
<td>140 FORMAT (150,150)</td>
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<td>1819</td>
<td>150 FORMAT (150,150)</td>
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<td>1820</td>
<td>140 WRITE (99,150) TN</td>
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<td>1821</td>
<td>150 FORMAT (150,150)</td>
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<td>150 FORMAT (150,150)</td>
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<td>1833</td>
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<td>1834</td>
<td>140 WRITE (99,150) TN</td>
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<td>1835</td>
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<td>1836</td>
<td>140 WRITE (99,150) TN</td>
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<tr>
<td>1873</td>
<td>150 FORMAT (150,150)</td>
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</tbody>
</table>

The updated code appears to be a subroutine for computing the appropriate bounds for a given damping factor, and it includes conditional logic for determining the correct bound based on the natural frequency and damping ratio. The code also checks for specific conditions, such as whether the damping factor is within a certain range, and handles those cases accordingly.
1932 C
1933 1934 C
1935 RATIO=SMAX/SMIN
1936 SRATIO=SNGLRATIO)
1937 WRITE(IUNIT.70) SRATIO
1938 70 FORMAT(' ',-RATIO OF LARGEST TO SMALLEST DIAGONAL STIFFNESS'.
1939 1  'MATRIX  ELEMENT IS.E10.3)
1940 RETURN
1941 ENO
1942 SUBROUTINE  SDFBAN(A,B,N,M,LT.RAT 10.DET.NCN,NSCALE)
1943 C
1944 C THIS  ROUTINE SOLVES SYSTEM OF EONS. AX=B WHERE A IS + TVE DEFINITE
1945 C SYMMETRIC BAND MATRIX. BY CHOLESKY'S METHOD.
1946 C LOWER HALF BAND ONLY (INCLUDING THE DIAGONAL) OF A IS STORED
1947 C COLUMN BY COLUMN IN A 1 DIMENSIONAL ARRAY.
1948 C SOLUTIONS X ARE RETURNED IN ARRAY B.
1949 C OPTIONAL SCALING OF MATRIX A IS AVAILABLE
1950 C N -  ORDER OF MATRIX A.
1951 CM-  LENGTH DF LOWER HALF BAND.
1952 C DETERMINANT OF A = DET *( 10* *NCN). 1 .E- 15<|DET|<1 E15
1953 C LT = 1 IF ONLY 1 B VECTOR OR IF FIRST OF SEVERAL. LT NOT = 1 FOR
1954 C SUBSEQUENT B VECTORS.
1955 C RATIO = SMALLEST RATIO OF 2 ELEMENTS ON MAIN DIAGONAL OF
1957 C NSCALE-0 IF SCALING NOT REOUIREO.
1958 C
1959 IMPLICIT  REAL'S (A-H.O-Z)
1960 DIMENSION  A(1).B(1)
1961 REAL  *8 MULT(200)
1962 1FIM.EQ.1) GO TO 101
1963 MM=M-1
1964 NM=N*M
1965 NM  1 =NM-MM
1966 C
1967 C  DUMMY STATEMENT INSERTED FOR COMPATIBILITY WITH ASSEMBLER VERSION.
1968 C  TRANSFORMATION OF A.
1969 C A IS  TRANSFORMED INTO A LOWER TRI ANGULAR MATRIX L SUCH THAT A-L.LT
1970 C  (L  T = TRANSPOSE OF L).  IF Y=LT X THEN L.Y=B.
1971 C  ERROR RETURN TAKEN IF RATI0<1.E-7
1972 C
1973 C IMPLICIT  REAL'S (A-H.O-Z)
1974 C DIMENSION  A(1).B(1)
1975 C REAL*8 MULT(200)
1976 C IF(M.EQ.1) GO TO 101
1977 C M- LENGTH OF LOWER HALF BAND.
1979 C LT=1 IF ONLY 1 B VECTOR OR IF FIRST OF SEVERAL. LT NOT = 1 FOR
1980 C SUBSEQUENT B VECTORS.
1982 C NSCALE=0 IF SCALING NOT REQUIRED.
1983 C
1984 C
1985 C
1986 C
1987 C  CONTINUE
1988 C  DO 3000 1=1,N
1989 C  DO 3000 1=1,N
1990 C  DO 3000 1=1,N
1991 C  DO 3000 1=1,N
1992 C  DO 3000 1=1,N
1993 C  DO 3000 1=1,N
1994 C  CONTINUE
1995 C  CONTINUE
1996 C  CONTINUE
1997 C  CONTINUE
1998 C  CONTINUE
1999 C  CONTINUE
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2054 C  CONTINUE
2055 C  CONTINUE
2056 C  CONTINUE
2057 C  CONTINUE
2058 C  CONTINUE
2059 C  CONTINUE
THE FOLLOWING STATEMENTS SOLVE FOR L+Y BY A FORWARDS AUGMENTATION.

HENCE FOR X FROM L+X+Y BY A BACKWARDS SUBSTITUTION.

IF SCALING OPTION USED, B IS SCALLED AND NORMALISED BEFORE SUBSTITUTION BEGINS.

55 SUM=0.00

THE STATEMENTS SOLVE FOR L+Y BY A FORWARDS AUGMENTATION.

HENCE FOR X FROM L+X+Y BY A BACKWARDS SUBSTITUTION.

IF SCALING OPTION USED, B IS SCALLED AND NORMALISED BEFORE SUBSTITUTION BEGINS.

55 SUM=0.00

THE STATEMENTS SOLVE FOR L+Y BY A FORWARDS AUGMENTATION.

HENCE FOR X FROM L+X+Y BY A BACKWARDS SUBSTITUTION.

IF SCALING OPTION USED, B IS SCALLED AND NORMALISED BEFORE SUBSTITUTION BEGINS.

55 SUM=0.00

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