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THE MODIFIED SUBSTITUTE STRUCTURE METHOD AS A DESIGN AID FOR SEISMIC RESISTANT COUPLED STRUCTURAL WALLS

by

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ABSTRACT

The modified substitute structure method is presented as a design aid which evaluates ductility requirements and deflections to determine the suitability of a reinforced concrete structural wall to withstand an anticipated siesmic disturbance. The procedure is analogous to elastic modal analysis but is an iterative technique which takes account of the stiffness loss of those members attempting to carry moments in excess of their ultimate moment capacity. Unlike the elastic modal analysis procedure, the method is capable of predicting the ductility demand in individual members of a given strength. This is the appropriate form of the problem for coupled walls.

The modified substitute structure method is applied through a computer program and the testing of this program is described. The effectiveness of the method for predicting ductlity demands and other parameters relating to structures undergoing inelasic behavior is evaluated by comparision with results obtained from the time step analysis program DRAIN-2D.

The modified substitute structure method is a procedure which is inexpensive to use and could be applied easily in a non-research design environment.

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CHAPTER 1 INTRODUCTION AND BACKGROUND

1.1 Coupled Walls as Earthquake Resisting Elements

The excellent behavior of structural walls both in earthquakes and under service load conditions has been reported in the literature from studies performed in far-ranging localities. While the news cameras and researchers have been photographing fallen and severely damaged ductile frames, examples of the good behavior of structural walls often goes unnoticed. Fintel[1] reports examples of the successful survival of structural wall buildings in earthquakes occurring in San Fernando, California (1971), Caracas, Venezuela (1967) and Skopje, Yugoslavia (1963). Despite the cases of excellent behavior, the structural wall system cannot be expected to behave well if the building is detailed in a manner that does not fully take account of the forces on the structure. Examples of this are the infamous Olive View hospital (1971) in which the first floor columns yielded before the structural walls above had a chance to act, and the Mount Mckinley apartments in Anchorage (1964) which suffered diagonal shear failures in the lintel beams of the coupled structural wall.

The numerous supporters of the structural wall system cite its beneficial energy absorption patterns and the way the system can cope with earthquake forces without undergoing large deformations which damage the often delicate non-structural elements and contents of the building. The structural wall system was originally thought of as a non-ductile system largely because of a series of seismic failures of improperly detailed walls. This led to the requirement of high 'k' factors when using static lateral load analysis of structural walled buildings. Research such as that performed by Paulay[5] has shown that it is possible to obtain ductility with proper detailing of the walls. The Canadian Standards Association building code realizes this by making special provisions to ascertain that the walls remain ductile. These provisions attempt to preclude non-ductility by making shear failure and other undesirable behavior more unlikely. However, while much of the research has dealt with the behavior of the walls, their capacities and ductile capabilities, the design of structural walls under dynamic loading has often been a somewhat irrational process.

The structural wall system is a dual load path system and it is important to appreciate the lateral load carrying methods of coupled structural walls if their analysis is to be understood correctly. A large proportion of the lateral load acting upon a coupled shear wall is taken essentially as two independant cantilevers would take the load by flexural bending. Compounding the situation, though, are the coupling beams which are bent in reverse curvature. Examining a freebody diagram of the distorted lintel beam shows that the moments causing the reverse curvature must be accompanied by a shear to maintain equilibrium. This shear produces an axial force in the walls, tension in one and compression in the other, in such a manner that it creates a couple which aids in counteracting the overturning moment caused by the lateral forces. The proportion of lateral load carried by each method is therefore determined by the member properties. As the lintel strength and rigidity increases, the resulting axial couple increases and the moment carried by the walls as individual cantilevers decreases. Making this situation even more complex is the inelastic behavior of the elements of the wall. The coupling beams, being subject to high reverse curvature over their short length, bear the brunt of this behavior and will be expected to pass well into their post-elastic range during a major seismic disturbance. While a member should not be expected to carry more than its ultimate moment it is up to the designer to ascertain that the excursions into the ultimate moment range will not result in large strength degradations. To achieve this the designer must know the levels of ductility demand that he can expect from the design earthquake acceleration of the structure. Therefore the design procedure must take account of the inelastic behavior of the coupling beams as they carry ultimate moment, but still undergo a deflection which is compatible with the remainder of the structure as it attempts to resist the earthquake forces in a similar manner. This sharing of the method of force-carrying illustrates how the design process must arrive at a proper relationship between the moment capacity of the walls, and the capacities, strengths and ductility demands of the members while at the same time giving an indication of the deflections to be expected.

From the energy absorption standpoint the system is excellent as the lintel beams are usually the first to undergo energy absorbing inelastic deformation while the walls which act as the main load carrying path act elastically. By their very nature as low load carrying members under normal circumstances and by virtue of their easily repairable location, the coupling beams provide a good place for energy absorption to occur without risking serious damage to the entire structure. This inelastic action of the coupling beams does not usually endanger the structure as the walls carry load in both axial coupling and flexural bending, the mechanism for their collapse can therefore only occur when both load paths are destroyed. For this to occur a series of hinges must form, one in either end of the coupling beams and one in each of the two walls.

The early belief that ductile frames were the ideal energy dissipating system is slowly losing favor. This reflects the growing belief that it is no longer sufficient merely to save the building during a seismic disturbance only to have it pulled down subsequently due to irreparable damage. The ductile frame is also losing favor as the displacements necessary for proper energy absorption will cause extensive damage to the contents and architectural finish of the structure which frequently exceeds the cost of the frame. Past earthquakes have shown that the ductile frame often absorbs energy in discrete locations instead of uniformly throughout the structure as anticipated. When this occurs the damaged locations frequently undergo more deformation than the designer would anticipate or desire. This was the case with the El Centro county office building in the 1979 earthquake in which large deformations occurred in the base of some of the columns requiring subsequent demolition of the building.

One advantage in the seismic design of structural walls over ductile frames is that the former will frequently behave more like the mathmatical model than will the latter. This is a result of the frequent neglect of the effects that partial infill walls and other 'cosmetic' building components will have on the behavior of frame structures. Some examples of this were seen in the Olive View hospital in which some effective column lengths had been reduced by architectural infill. This has the effect of concentrating any inelastic action into a shorter section of the column as well as increasing the shear force in the member. It is far less likely that the behavior of a wall will be accidentally modified.

1.2 Purpose of This Thesis

The analysis of structures for the purpose of seismic design can be done with various levels of sophistication:

(i) For small structures a quasi-static analysis using the equivalent forces defined by a building code is an appropiate procedure.

(ii) For medium size structures-for example, residential buildings in the 10 to 25 story range--an elastic modal analysis based on a design spectrum is generally used.

The root-sum-square forces from this analysis are then divided by the available structure ductility associated with the particular structural system, to give the yield level forces for which the building should be designed.

(iii) For larger or more complex structures, an inelastic time step analysis based on an appropiate earthquake record or records should be applied.

The procedure described under (ii) above is applicable to the case of frame structures, where the available ductility associated with the structural system is known and the yield level moments are the desired quantities. In the case of residential buildings consisting of coupled structural walls however, the procedure is not really applicable. In these buildings the coupling beams are generally slabs or short lintels of minimum cross-sectional dimensions--typically 18 inches deep by 8 inches wide. It is not possible to reinforce such members in the manner indicated by Paulay to give the optimum levels of yield moment and ductility. Instead, one can reinforce the members to give the maximum possible moment and then analyse the system to see whether the ductility demand and shear capacity of the members can be met. If it cannot, some change in the structural layout is required.

To repeat: in the case of frames, the desired ductility level is known, and the corresponding yield strength is required; the linear elastic spectral analysis described under (ii) above gives the desired result. In the case of coupled walls the maximum available strength is known, and the ductility demand is to be evaluated to test the structural layout. The aim of this thesis is to provide a method of doing this for smaller structures, for which a fullscale inelastic analysis is not feasible from an economic or design time point of view.

1.3 Examination of Structural Analysis Methods for Seismic Design

Before describing the proposed method, it is worthwhile to point out the faults of the present methods of analysing coupled walls. They fall into three broad catagories. The first of these involves a code specified static lateral load applied to various models of the structure including a 'laminar' model in which the properties of the lintels have been smeared throughout the height of the structure. This suffers from not fully reflecting the dynamic nature of load and structure. The second category is the elastic modal analysis method which has been used many times in practice for earthquake design but has the disadvantage that it does not reflect the considerable effects that the inelastic behavior will have on the the structural wall. This method also does not predict the ductility demands of the wall elements. The final category is the time step inelastic method. Its handicap is that it is expensive to use and would frequently only be applied to larger buildings in circumstances where an outside consultant is brought in for the earthquake analysis.

The advantages of inelastic programs to model the behavior of buildings under earthquake loads have been known for years. Most of the arguments put forward in their favor extole the virtues of being able to determine much better the true performance of the structure as well as the ductilty demands of the members. While these points are valid, the programs that have been used mostly to date to model inelastic properties are time step programs. Though these programs can frequently reproduce the effect of a given earthquake on a given structure they do so at a cost that is often prohibitive for many structures. There are two reason for this: the first of which is of great practical significance -- time step programs are not cheap to run. There will naturally be a necessity for several computer runs as the structure is altered to iterate in on the required strength and stiffness, and also to meet the demands of the architect and owner. For a successful inelastic analysis of a structure utilizing time step methods, the mathematical model of the structure must undergo testing with a variety of appropriate earthquakes if it is to be designed properly. Testing with a selection of earthquakes is the only way that time step analysis can reflect the uncertainty associated with the motions of a future disturbance. This item itself necessitates several runs. Time step analysis programs may also require an initial run on a modal analysis type program in order that the frequencies can be determined to input to parameters of the time step damping. This also increases the cost of program use.

The other expense of time step analysis programs is that they are somewhat removed from the realm of the average structural engineer. The availability and input requirements implies that they will only be considered in somewhat specialized consulting situations. This also implies that the earthquake analysis of the 'average' structure will be conducted using at best elastic modal analysis or code specified lateral loads applied to an elastic structure. While these elastic methods have much merit in their own right, large earthquake disturbances present a violently non-static load on a structure during which very few buildings can be expected to remain totally in their elastic range. Failure to include these inelastic actions in the design analysis procedure is a serious drawback when considering the structural wall.

Early studies of coupled shear walls attempted to model their properties by replacing the discrete coupling beams by continuous laminae. The technique was advanced to take account of inelastic behavior but still had difficulty in dealing with walls with properties that were not constant over the height of the structure. The method also suffered from the objection that it was not one that takes account of the dynamic response of the structure. Articles demonstrating the method showed their loading as a static, often triangular load acting on the wall with only one deflected shape considered, that being the one that could be described as approximating the first mode of the wall. The omission, in an earthquake analysis method, of the dynamic interaction between the load, structure and response is too great a simplification to be accepted when methods exist that do take account of the problem.

It has been fortunate that the increase in electronic computational capabilities have increased at the same time that our necessity for rational earthquake analysis has increased, for without the computer the task verges on the impossible. This is especially the case if inelastic and dynamic effects are taken into account during the analysis. During the progress of research on this thesis a computer program was developed by modification of a program written during some earlier research on the Modified Substitute Structure Method by Sumio Yoshida[10].

In developing a computer program which could apply the method to a structure, considerable effort was applied to make it one that could be used by the practising engineer for building design. The data input, while possibly varying in format from other existing static analysis programs, does not demand input that is greatly different either in type or amount from one of those programs. The engineer who can apply a static lateral load to a computer model of a structure will not find it much of a task to determine the ductility requirements and inelastic deflections used in this method.

A very important advantage that comes with the increased availability of computer aided earthquake design is that it allows a wider range in the size of structures that can economically be considered rationally for earthquake loadings. As the cost of executing the programs decreases, it becomes practical economically to include in the design of the cheaper structure a more complete consideration of its dynamic characteristics than is permitted with the lateral load method.

Another important advance in the field of earthquake engineering is our improving ability to predict the expected intensities of ground disturbances at a given location. This has resulted from a combination of improved and more numerous measuring stations and better computational means to interpret the recorded data. At present the abililty to predict exactly the time and motions of a disturbance is nonexistant. Yet it is possible for the seismologist to make good estimates of both the maximum acceleration and spectral content that can be expected in most locations. Time step analysis makes use of this information by finding earthquakes which approximate the anticipated spectrum, scaling these to the predicted acceleration and applying them to a computer model of the structure.

For elastic analysis the problem of choosing appropriate earthquakes was circumvented by the use of modal analysis based on the design spectrum directly. It was soon realized by those using both schemes that the modal method had several other advantages in terms of savings of computer time and ease of programming. Indeed, if a computer program is available for performing structural analysis using the stiffness matrix method, and if this is used on a system that has an eigenvalue finding routine, then it is a fairly simple problem to combine the two to produce a modal analysis program.

While the modal method has been quite widely used for elastic analysis, inelastic dynamic analysis has had to rely on time step analysis programs which examine the state of the structural members at discrete intervals of the excitation period, to determine strength and stiffness degradation using idealized hysteresis diagrams. What the Modified Substitute Structure Method does is extend the modal method with all its inherent advantages, into the inelastic range.

No numerical technique is exact: there always remains a tradeoff between the complexity, and therefore the execution cost of the method, and the desired accuracy of the answer. It must be realized that the input properties to most structural and especially earthquake analyses are subject to variation and experimental error. It makes little sense to become agitated over differences in the second or third figure of a displacement or ductility value when the input acceleration is at best accurate in only its first figure. It also makes little sense to achieve this extra accuracy when it requires an order of magnitude cost increase. What is important in a numerical method to be applied in a design situation is that it give good, reasonable answers, and that it can be used to predict the direction selected changes in the structural or excitation properties will have upon those results. While in some places the present work makes comparisons with those results obtained from inelastic time step analysis, it is done not in the belief that they present the indisputable truth in terms of the behavior of a structure under earthquake loads, but rather that the method is presently accepted as one of the better numerical analysis techniques that can be applied to the problem.

1.4 Scope

This thesis proceeds by describing the Modified Substitute Structure Method, its development and limitations. It then moves on to discuss some of the improvements, developments and observations made while attempting to modify and apply a computer program to analyze structural walls using the method. The testing of the program for elastic modal analysis is then discussed, partly because this process took far longer than expected and resulted in some unexpected changes being made to the program. Following this is perhaps the chapter of most significance and concern in which the tests of the program's inelastic capabilities are related. This involves a two step demonstration in which it is shown that, firstly, the ductility demand patterns such as those reported by Paulay[4] can be predicted; and, secondly, that the numerical values consistant with time step analysis methods can be reproduced satisfactorly. The application of the method to the analysis of a sixteen-story structural wall in a design example acts as a further test of the method and is related in a chapter of its own as are the conclusions which follow.

It is hoped that in reading the pages that follow, researchers and engineers will be able to see a design method that can be applied to structures taking account of inelastic behavior and the dynamic nature of both earthquake and structure, in a rational, safe, yet easily applied manner.

CHAPTER 2 INTRODUCTION TO THE SUBSTITUTE STRUCTURE METHOD AND THE MODIFIED SUBSTITUTE STRUCTURE METHOD.

The modified substitute structure method is a numerical method closely akin to modal analysis but extending that technique into the inelastic range. The method and its developments are discussed here so that the reader can gain a better insight into the application to structural walls.

2.1 The Substitute Structure Method.

As can be expected from its title, the modified substitute structure method was developed from adaptations made to the substitute structure method and an examination of this earlier method can give insight to the later one.

The substitute structure method was proposed by Shibata and Sozen[8] as a design procedure for reinforced concrete structures which could be used to establish the yield forces that should be provided for in the design, assuming that the initial stiffness and available ductility are known. This is generally the case by the time aseismic design is approached. The method was developed as a means of establishing the member properties necessary to achieve an acceptable structural response under earthquake loading.

As with any technique the method is subject to some restrictions which define the type of problem to which it is applicable. For the substitute structure method, these are as follows:

(1) The system must be capable of analysis in one vertical plane. This limits the method to plane frame analysis and eliminates problems which involve torsion and biaxial bending.

(2) The structure must be one in which abrupt changes in geometry or mass do not occur over the height. This limits the analysis to regular-shaped structures. Although it is possible that some structures outside this class could be analyzed with success, they are probably the exception and all structures not of this class should be eliminated. Systems failing to meet this restriction frequently cause problems in dynamic analysis regardless of the technique in use.

(3) Columns, walls represented as columns, and beams may be designed with different damage ratios but all beams in a given bay or the columns on a given axis should have the same value. Just why this is a criterion for the substitute structure method was not explained in the original papers concerning the method, but it may well have been imposed simply because only structures of this type were tested by the original authors. As will be shown later this restriction is not necessary in the modified substitute structure method.

(4) All structural elements and joints must be reinforced to avoid significant strength decay as a result of repeated reversals of the anticipated inelastic displacements. It is assumed in the method that the stiffness of the members involved will be reduced when they yield and stiffness losses are calculated on the basis of given 'damage ratios'. What the method does not allow for is a failure of the member before it reaches the specified ductility; the responsibility for selecting this ductility lies with the designer. It should also be noted that it is presumed that members do not fail in shear or buckling before reaching the desired flexural load. This necessary characteristic of any aseismic structure is again the responsibility of the designer.

(5) The non-structural elements must not interfere significantly with the dynamic response of the structure. This is an obvious restriction applying to any method of dynamic analysis in which special elements have not been included in the model to account for items such as infill walls.

For many simple structures these five restrictions are not a serious drawback and the method provides an inexpensive design aid.

The steps involved in the method will be described briefly here and are also shown in the flowchart of figure 2.1. Before starting to use this method it is assumed that the designer, through evaluation of the wind and gravity loads, and aided by experience, has already determined the gross sizes of the concrete members involved. It is also assumed that a smoothed response acceleration spectrum has been obtained for the design earthquake, and that the designer has chosen tolerable 'damage ratios' for the members.

The first step is then the evaluation of the stiffness of the members on the basis of their expected 'damage ratios'; this is done through the following formula:

$$(EI)_{si} = \frac{(EI)_{ai}}{\mu_i} \qquad (2.1)$$

where $(EI)_{ai}$ is the stiffness of the element in the real structure

$(EI)_{si}$ is the stiffness of the element in the substitute structure

μ_i is the damage ratio of the element.

The concept of 'damage ratio' is central to the application of the method: it is comparable to ductility; but while curvature ductility is the ratio of the curvature of the member under the applied moment to the curvature at yield moment, the damage ratio is a number designed specifically to give the equivalent linear member stiffness, which may be used as though the moment were linearly related to curvature from initial to final load. The damage ratio gives a stiffness by formula 2.1 which implies that under equal and opposite end moments an end rotation of $\mu_i \theta_y$ would be achieved where θ_y is the rotation at the onset of yielding. Under these conditions, where the end moments of the beam under consideration are equal, and where the moment-rotation curve for the real beam is truly elasto-plastic then the numeric values of damage ratio and curvature ductility for the member will be equal. While not giving any real indication of what values should be used in general for the damage ratios, Shibata and Sozen in their analysis used a value of 6 for the beams and 1 for the columns for those structures with flexible beams.

Knowing these 'substitute' stiffnesses and other structure information such as joint locations and member lengths, the structure stiffness matrix is constructed. A mass matrix with the masses concentrated at the joints, which leads to a diagonal mass matrix and dynamically uncouples the response equations, must also be constructed. From mass and stiffness matrices, the modal frequencies and mode shapes are determined as usual from the following equation:

$$\left| [K] - \omega^2 [m] \right| = 0 \qquad (2.2)$$

where (K) is the structure stiffness matrix

[m] is the mass matrix

and ω is the angular frequency.

With the angular frequencies evaluated from equation 2.2 the value of the spectral acceleration can be determined from the response spectrum for the chosen earthquake. In the manner of standard modal analysis the matrix of applied seismic forces (F^r) is now calculated for each mode by use of the following formula:

$$(F^r) = (A^r) \left\{ \frac{(A^r)^T [m] \, (I)}{(A^r)^T [m] \, (A^r)} \right\} S_a^r \, [m] \qquad (2.3)$$

where (A^r) is the r th mode shape vector and superscript T denotes transpose.

(I) is the identity matrix

and (S_a^r) is the spectral acceleration for the r th mode computed from the zero damped design spectrum using the natural frequency of that mode.

In the above formula the expression in the curled brackets is frequently called the modal participation factor and is calculated separately. At this stage in the procedure the forces on the structure have been calculated and it is now necessary to compute the resulting displacements; these are calculated using the standard stiffness method.

From the member forces arising from the seismic loads, in particular the member end bending moments, the smeared damping ratio is computed for each mode. The damping factor for the individual members is calculated first using a formula from laboratory tests by Gulkan and Sozen published in 1974, which follows:

$$\beta_{si} = 0.02 + 0.2\left(1 - \frac{1}{\sqrt{\mu_i}}\right) \qquad (2.4)$$

where β_{si} is the substitute damping factor, a value of viscous damping to represent the hysteretic energy dissipated by the member.

The formula was developed to adjust the analytic results of one-story frames analyzed by means of a linear spectrum to match more closely the observed experimental behavior of the frames.

A single damping value is required for each mode so that the damping ratios for the individual members must be combined to form a composite value for the structure. This 'smearing'of the structure damping is based on the flexural energy of deformation of the members, computed by the following formula:

$$P_i^r = \frac{L_i}{6(EI)_{si}} \left[(M_{ai}^r)^2 + (M_{bi}^r)^2 - M_{ai}^r M_{bi}^r \right] \qquad (2.5)$$

in which P_i^r is the energy of deformation for element i

M_{ai}^r and M_{bi}^r are the moments at the ends of substitute frame element i, for the r^{th} mode.

Using this formula for the flexural energy of deformation of the individual members, the smeared damping ratio is expressed as:

$$\beta_r = \frac{\sum_i (P_i^r \beta_{si})}{\sum_i P_i^r} \qquad (2.6)$$

where β_r is the ratio of critical damping for the r^{th} mode.

This procedure gives unique damping ratios for each mode. The smeared damping ratio receives its major contributions from those members with the largest element damping ratios and those members with the largest bending moments, two groups which do not necessarily coincide.

With the damping known for each mode the solution is recalculated. As no revision has been made to the damage ratios the structure stiffness matrix remains the same, as does the make-up of the mass matrix. With damping values in the eight to fifteen percent range common to concrete frames, the mode shapes and frequencies do not change and therefore do not need recalculating. What does change, however, is the acceleration force calculated from the response spectrum. Hence formula 2.3 must be re-evaluated, producing member forces which differ from the initial values for the undamped structure. As only one 'iteration' is performed it is unnecessary to recalculate damping ratios from the new forces.

The member forces which have been calculated for each mode are now combined in the usual manner by the Root-Sum-Square (RSS) method, with a modification suggested by by Shibata and Sozen: they multiply all the forces by a common factor which will increase them if the magnitude of the two largest contributors are similar. This reflects the higher probability of coincidence of the maximum modal forces in any two modes compared to their probable coincidence in several modes. The multiplying factor is determined using the base shear of the structure in the following formula:

$$(F_i) = (F_{iRSS})\,\frac{V_{RSS} + V_{ABS}}{2V_{RSS}} \qquad (2.7)$$

where: (F_i) = the ith design force

(F_{iRSS}) = the Root-Sum-Square force

V_{RSS}= the RSS base shear

V_{ABS}= the maximum value of the sum of the absolute values of any two base shears.

This factor will increase all the design forces by slightly over twenty percent in the cases where only two modes are analyzed and they have equal base shears. Under any other condition it will increase the RSS forces by between zero and twenty percent.

The final step in the substitute structure method is to increase the design moments pertaining to the columns by twenty percent to prevent the undesirable results of plastic hinges in these members. Thus the final aim of the substitute structure method is achieved: the design forces for seismic loading are produced.

2.2 The Modified Substitute Structure Method.

Although the substitute structure method was intended explicitly as a design method and not an analysis method, the modified substitute structure method was developed for the analysis of existing reinforced concrete buildings. This was done to predict the extent and location of damage for 'retrofit' purposes.

In this method the input data differs from that of the substitute structure method in that the yield moments, which presumably would be known for the members of an existing structure, are read in as part of the input data, together with initial stiffnesses. The damage ratios are the sought for quantities. During the execution of the method the members are not allowed to carry moments which exceed the specified yield moments. Much the same procedure is used as in the substitute structure method, but the technique is an iterative one.

The structure stiffness matrix is set up in the same manner using the damage ratios to modify the member stiffnesses; though these may be set to unity for the first iteration. An alternative procedure involves the designer estimating the damage ratios for the members prior to the analysis; although this does not affect the final damage ratios produced it will often reduce the number of iterations that are performed before convergence is achieved. Eigenvalues and eigenvectors are then calculated to find natural frequencies and mode shapes as before. During the first trial the smeared damping values accounting for hysteretic energy loss are unknown, so the member forces are calculated using 'appropriate' damping values which can be specified by the program user, instead of a calculated value. Subsequent iterations use the same procedure as the substitute structure method to calculate the damping ratios. Knowing the damping ratios, revised forces and displacements are computed, as well as root-sum-square forces. Those members whose RSS moments exceed yield have their damage ratios modified according to the following formula:

$$\mu_{n+1} = \mu_n \frac{M_n}{M_y} \geqslant 1 \qquad (2.8)$$

where: μ_{n+1} is the damage ratio for the n-1 iteration

μ_n is the damage ratio for the nth iteration

M_n is the larger RSS end moment from the nth iteration

M_y is the yield moment for the member.

The limit of unity is set since those members that have not yielded clearly still have the initial stiffness. The final two steps from the substitute structure method are omitted: there is no increase in the RSS forces to account for concidence of modes and the moments in the columns are not increased by twenty percent. The elimination of these two steps reflects the difference in philosophy when the procedure is used for analysis rather than design.

With the new damage ratios , and the smeared damping ratios from the previous iteration, another iteration is performed, commencing with the calculation of a new stiffness matrix and finishing with a further refined set of damage ratios. When all the member forces are either below or within a tolerable limit of their yield value, the cycling is halted. At this stage the damage ratios that have been determined by iteration are printed. A diagramatic demonstration of the program steps can be seen in flowchart form in figure 2.2.

Although the program ends with the printing of the calculated damage ratios, the final step required is an interpretation of the output. In the retrofit procedure for which this method was originally intended, this involves the engineer's determination of the acceptability of these ratios in relation to the detailing of the structure under analysis.

Although most of the restrictions which apply also to the original substitute structure method apply to the modified method, there are some other simplifications which are accepted in most computer analysis of structures. Beams and columns are modelled as line members, the P-delta effect is ignored and for purposes of diagonalizing the mass matrix, the structure mass is assumed concentrated at the nodes. Only one mass per floor seems to be necessary or desirable. Members involved in the analysis should be symmetric as the damage ratios are based only on the largest root mean square moment on the member concerned and no differentiation is made between positive and negative bending moment. Changing axial and shear forces are not considered in determining the yield state of the members. Account is taken of axial shortening generated from the lateral earthquake loads, but the static forces that would be generated by the dead weight or other gravity loads are not considered either in the determination of the damage ratios or of the root mean square forces.

One of the chief advantages of both the substitute and the modified substitute structure methods over time step analysis is, as already mentioned, the use of a smoothed response spectrum. While discussing restrictions on the methods it is perhaps worthwhile to discuss the restrictions that are and are not placed upon this spectrum. The use of a linear spectrum is unnecessary. Shibata and Sozen[8] give as one of their requirements for the substitute structure method that any increase in period results in a decrease in the spectral acceleration. In their three test structures this is the case for at least the fundamental mode.

The modified substitute structure method removes any restrictions imposed by requirements of the spectrum, to a large extent, through its iterative procedure. In Yoshida's thesis a spectrum involving fifty increments was used in tabular form for some of the runs. Although it was found that the damage ratios did not converge without some oscillation and up to a 100 percent increase in the number of iterations was required, a successful convergence was found in the trials. These tests, while not showing that better results could be obtained by using a non-linear response spectrum, did prove that such a spectrum was not an impediment to convergence of the damage ratios.

To determine the applicability of their methods Shibata and Sozen in their paper, and Yoshida in his thesis, used time step dynamic analysis programs on the structures for which the methods had been used. Shibata and Sozen, when testing the substitute structure method used three one-bay test frames with a height ranging from three to ten stories. Their method of testing was to find the design forces using the substitute structure method, then to design the frames on the basis of these forces. The frames were then analysed using the time step analysis program SAKE and a comparison of the damage ratios so obtained with the initially specified values was made. The results were favourable for all three frames where the design forces had been calculated on the basis of a damage ratio of six for the beams and one for the columns. For the ten-story frame, while the column values showed some scatter, with only three of the ten stories predicted conservatively, the beams had an average damage ratio of 5.5 and were all conservative. The five-story frame had only one damage ratio larger than unity in the columns, while the beams averaged a damage ratio of 4.6, and all were below the design value of 6, which was therefore conservative. The three-story frame produced the best results with all average damage ratios found in the time step analysis being close to but below the values chosen when doing the substitute structure analysis.

As was expected in using a design spectrum that comprised four earthquakes (in the case of Shibata and Sozen's design spectrum 'A': El Centro E.W., El Centro N.S., Kern County S.69E., and Kern County N.21E.), some records produced damage ratios and displacements that were considerably above the average while others were below. To design a structure so that damage ratios should be below the specified values for spectra corresponding to all earthquakes scaled to a given acceleration would produce an overly conservative design.

As a test of the Modified Substitute Structure Method, Yoshida tested four structures under the same spectrum 'A' as that used by Shibata and Sozen. These structures offered a variety of structural configurations corresponding to small and medium structures. They were: a two-story, two-bay frame; a three-story, three-bay frame; a six-story, one-bay frame; and a six-story, three-bay frame. For comparison purposes the damage ratios were calculated by time step analysis using the program SAKE, with the records of the four individual earthquakes that had gone into the spectrum. The comparison showed very favourable results in all cases. The CPU time reduction for the modified substitute structure method ranged from over one hundred seconds in the case of the largest structure (120 sec for the time step analysis as opposed to 2.3 sec for the proposed method) to eleven seconds for the smallest structure (12.1 sec to 0.91 sec).

To summarize Yoshida's results they can be regarded as giving an excellent indication to a designer of 'trouble spots' in his structure. The three-bay, three-story structure showed the best results with all members except three within fifteen percent of what would be predicted by time step analysis. The three members outside this group were all columns on the top story; their damage ratio was predicted conservatively by the method. The two-bay, two-story structure showed excessive yielding in the bottom story columns in both analysis procedures but the method did not predict as much yielding here as did time step analysis. All other members in the structure were within fifteen percent or conservatively predicted. The three-bay, six-story frame showed all members within thirty percent of the true value or conservatively predicted, over half of the members were well within fifteen percent of the average for the non-linear analysis. The six-story, one-bay frame, when analyzed by the modified substitute structure method, showed numerical results which predicted excessive yielding throughout the structure, but did not produce a close numerical forecast of the damage ratios. It was concluded that the method was a poor numeric predictor in cases where there was extensive and excessive yielding of the members throughout the structure. It should be noted that in the test structures, the columns of one line or the beams of one bay sometimes did not have equal capacities. Although the substitute structure method was restricted by Shibata and Sozen to structures which did have equal capacities in these circumstances these tests show it not to be a necessary restriction for the modified substitute structure method.

Our present knowledge of predicting the exact excitation pattern of a future earthquake at a given site is at best limited. The spectrum approach makes concessions to this by using an envelope of effects from past events,thus expressing the future earthquake in a more general manner than can be considered when using directly the individual excitation records of former earthquakes. The modified substitute structure method has been shown to offer the designer a good alternative to time step analysis for prediction of the damage ratios in reinforced concrete frame members. The method becomes even more attractive should the designer wish to design his structure on the basis of 'mixing' the excitation results from several past earthquakes to better estimate the damage ratios caused by future seismic events. While the method has been found effective for normal reinforced concrete frame elements it is the purpose of this thesis to examine the effectiveness of the method when applied to structural walls.

CHAPTER 3 ALTERATIONS TO THE METHOD FOR THE ANALYSIS OF STRUCTURAL WALLS

Under its original formulation the modified substitute structure method was intended to be used in the analysis of reinforced concrete frames. This chaper discusses the changes made to the method to adapt it to the analysis of structural walls.

3.1 CONVERGENCE SCHEMES

As with any iterative procedure, some criterion must be used for determining when the solution has reached a level of accuracy such that the process can be halted. For the modified substitute structure method this criterion can be based on either a maximum change between the damage ratios of successive iterations, or on the closeness of yielded members to their moment capacity. By examining the formula for modifying the damage ratios at the end of each iteration, it can be shown that for a member which remains above a unit damage ratio, the damage ratio at the end of the nth iteration is given by the following formula:

$$\mu_n = \mu_i \left(\frac{M_1}{M_{CAP}}\right) \cdot \left(\frac{M_2}{M_{CAP}}\right) \cdot \left(\frac{M_3}{M_{CAP}}\right) \cdots\cdots \left(\frac{M_n}{M_{CAP}}\right) \qquad (3.1)$$

where

M_n is the largest end moment for the member at the end of the nth iteration and M_{CAP} is the bending moment capacity of the member.

During the progress of the iterative procedure, if the damage ratios are to converge, the ratio of the member end moment to capacity must converge to unity. The original convergence criterion of the modified substitute structure method was deemed to be achieved when none of the members with damage ratios above unity were outside a specified tolerance from their capacity. This deviation of the damaged members from their capacity is referred to here as the bending moment error.

To ensure that the damage ratios converged, a very strict tolerance was imposed on the bending moment error requiring the maximum moments carried by the members to be almost exactly the capacity of the member. These tolerances were in the order of 10^{-3}, implying that damaged members should be within a tenth of a percent of their capacity. In practical terms this is an excessively small tolerance to place on the moments. With variations in member and material properties it is unlikely that member capacities would be known to anywhere near this accuracy. It was observed during some runs that damage ratios, when converging to meet this criterion, would often vary only in the second or third decimal places during all but the initial iterations. As damage ratios cannot, under even the best circumstances, be regarded as more 'accurate' than a single decimal place, these extra iterations are unnecessary. Although the CPU time for the modified substitute structure method is dependent upon the degrees of freedom of the structure and the half-bandwidth of the banded stiffness matrix, it is most heavily influenced by the number of iterations to achieve convergence and any saving of unnecessary iterations will be reflected in a saving of computation costs. For this reason a revision of the original convergence criterion was undertaken.

The revised convergence criterion was based on two conditions rather than one. The first of these was to require that the bending moment error be less than five percent of the member capacity for all the members of the structure. This is a radical change from the previous criterion of a tenth of a percent on this error. The second convergence criterion was to require that the largest change in damage ratios between successive iterations be one percent. This last condition was overridden, in the case of members with damage ratios of less than five, which would be unduly refined by this requirement. In the case of these smaller values the criterion was that the absolute difference in the ratios for the individual members be less than 0.1. In algebraic terms the new convergence criteria can be described as follows:

For convergence both 3.2(a) and 3.2(b) must be satisfied.

$$\text{if} \quad \mu_n > 1 \qquad \left| \frac{M_n - M_{CAP}}{M_{CAP}} \right| < 0.05 \qquad (3.2a)$$

$$\text{if } \mu_n > 5 \qquad \left| \frac{\mu_n - \mu_{n-1}}{\mu_n} \right| < 0.01 \qquad (3.2b)$$

$$\text{if } \mu_n \leqslant 5 \qquad \left| \mu_n - \mu_{n-1} \right| < 0.1 \qquad (3.2b)$$

The results of using these revised convergence criteria will be discussed after a technique is introduced to further save unnecessary iterations - a convergence speeding routine.

3.2 CONVERGENCE SPEEDING ROUTINE

In early computer runs using the modified substitute structure method it was observed that for some structures, the damage ratios would converge very slowly to the final answer or oscillate around this point. The original modified substitute structure method program contained a routine which proved effective in arriving at the final answer for those cases in which the changing damage ratios were either decreasing or increasing steadily. This routine operated on the basis of adding to the damage ratios that were to be returned to the main program a factor multiplied by the change in the damage ratios over the last iteration. In this manner, changing damage ratios were moved faster in a direction which hopefully was toward the true answer. The routine achieved good results in many cases by cutting down considerably the number of iterations while still achieving the same solution upon convergence. Unfortunately, in those cases where the damage ratios were oscillating at each iteration the routine actually was a deterrent to convergence. In these cases the iteration procedure usually continued until the maximum number of iterations had been exceeded. The solution to this problem was to better establish the damage ratio trends by keeping track of damage ratios from more than just the last iteration. However, practicality dictates that storage of all damage ratios is undesirable. For the case of a coupled ten-story structural wall with a maximum iteration count of two hundred, this would require an array to store a possible six thousand damage ratios. The required array space would rapidly become larger as the number of stories or coupled walls increased. Just exactly what to do with this potentially vast collection of damage ratios when stored would also be a problem of considerable proportion.

The adopted solution to these problems was a convergence routine which stored and used the damage ratios from the past two iterations. Hence, three values are known, these being the damage ratio produced in the current iteration and those from the previous two iterations. What occurs in the routine is a possible modification of the latest damage ratio before returning it to the program. By rotating the ratios by discarding the oldest value during the iteration procedure, a minimal amount of extra array space is required. During the deployment of the routine, by executing a maximum of two 'arithmetic if' comparisons, the nine possible trends in the ratio can be determined. In this manner those ratios that seem to be consistently decreasing or increasing can have the damage ratio modified by appropriately adding or subtracting a factor multiplied by the difference of the last two values as in the original program. On the other hand, in the case of oscillating damage ratios, the oscillations are damped into producing an answer lying between the last two values. It was decided that in those cases in which the ratios did not change for two consecutive iterations, no modification should be made by the convergence speeding routine. A more detailed view of the workings of this routine can be obtained by looking at the flowchart of the routine, shown in figure 3.1, which also shows in schematic form, the nine possible cases for the relative positioning of the three damage ratios. From the flowchart it can be seen that the routine is controlled by a factor beta (β), which is a positive number less than unity. A value of zero for beta effectively shuts off the convergence speeding routine. This is done during the first few wildly changing iterations to let the modified substitute structure method program naturally home closer to the final answers.

As the new convergence routine was formulated to work with the revised convergence criteria both were included in the modified substitute structure method computer program before further testing was carried out. Hence any reduction in the number of iterations required cannot be solely attributed to either the new routine or the revised criteria. Six tests were made on various structures which had been run already under the original convergence scheme. During these tests the value of beta in the new convergence speeding routine was kept at an arbitrarily chosen value of 0.25. Without exception the results showed a considerable decrease in the number of iterations required to achieve convergence. Some showed a decrease of nearly eighty percent and others a more modest twenty percent.

To determine the validity of the procedure a comparison must be made between the results obtained for damage ratios produced under the new and old convergence schemes. In those cases in which the number of iterations had been small (i.e. less than about fifteen) under the old convergence scheme the new method produced almost identical results, changing only insignificant figures in all quantities of concern. One would hope that successful convergence criteria would only change the insignificant decimal places as the tolerances were made more strict. Indeed the initial results show this to be true in those structures for which time step and modified substitute structure method answers have previously been closest.

Those structures that had difficulty converging under the original convergence scheme showed this trend again under the new scheme. The results of the six-story frame under the two schemes are shown in figure 3.2. Although the results are somewhat different under the new criteria they still reflect the trends that emerge from the time step analysis runs performed by Sumio Yoshida.

The value of beta used to obtain the convergence in the previous results was set at 0.25. This number has been chosen quite arbitrarily but meets the criterion as it lies between zero and one. To determine an optimal value of beta for all structures that could ever be considered is beyond the scope of this work. As different structures converge upon their final answer in a variety of ways a 'best' value of beta will not be uniquely defined. To test the effect of varying beta on one structure, the tolerance demand of the bending moment error was temporarily altered to make convergence more difficult and to accentuate the effect of the convergence speeding routine. Several computer runs were then made on a six-story structure using different values of beta to achieve convergence. The damage ratios at the end of each iteration were then plotted for one of the members of the structure as a function of the iteration number. This graph can be seen in figure 3.3. Examination of this figure shows that as beta approaches one the convergence is accelerated. The final value of the damage ratio is independent of the value of beta as long as beta lies in its admissible range. As was expected, a value of beta in excess of one causes divergence. Examination of this graph and the reduction in execution times for the cases studied leaves no doubt that the new convergence scheme is a viable method of reducing the required number of iterations and saving CPU time.

3.3 THE EFFECT OF USING ZERO SMEARED DAMPING RATIO AT THE START OF EACH ITERATION

As has been outlined in an earlier section dealing with the theory of the modified substitute structure method, each iteration involves two major sections. To review: first the natural frequencies of the structure are found, using zero damping, to obtain inertia forces from the spectrum. These forces are then applied to the structure, giving the internal member forces and a smeared damping ratio. The second major iteration step is to use this smeared damping force to recalculate the member forces and hence the new damage ratios. The use of the zero damping ratio at the start of all iterations, is a vestige of the substitute structure method in which, without an iterative procedure, any better estimate of the smeared damping ratio is a guess. In the modified substitute structure method one knows approximate damping ratios for the different modes of the structure by looking at the ratio determined in the last iteration. It was briefly thought that convergence would be improved by using these latest values when calculating the spectral acceleration for determination of the smeared damping ratio of the current iteration. This would replace the process of returning to zero damping for the first half of each cycle of the iteration.

This procedure was adopted in test runs using those structures which had undergone testing in Yoshida's thesis. The three-bay, three-story test structure was run as was the six-story, one-bay structure. Convergence criteria and schemes for these tests were in all cases those of the original modified substitute structure method. Surprisingly, the number of iterations required for both structures remained precisely the same. The three-story structure still required thirteen iterations and the six-story frame required sixty three before achieving convergence. When the answers were examined it was found that the damage ratios varied only in the third decimal place and therefore insufficiently to be of concern. Hence it was decided to refrain from zeroing the smeared damping ratios at the start of each iteration and to use those already in the damping array. As the damping matrix is small this achieves only a minimal saving in storage and execution requirements.

After eliminating the necessity to repeatedly zero the damping matrix, the next step in this evaluation was to compute the damping twice in each iteration: once from the initial pass (which previously was the zero damped pass); then subsequently from the pass in which the damped forces were applied. Giving the subsequent iteration a more 'accurate' damping ratio to start was thought to lead to a quicker convergence. In Yoshida's original method the damping had not been calculated in the second pass of each iteration as it serves no purpose if this damping ratio is not to be used in the following iteration. After modification of the computer program the same two structures used above were retested to determine the effect of this measure. The results showed similar trends to those of the previous tests, for while a few insignificant decimal places had been changed, the number of iterations remained exactly the same. It was concluded, therefore, that the second calculation of the modal damping values at the end of an iteration was not a worthwhile use of CPU time and was as unnecessary as zeroing the modal damping values at the start of each iteration. Remodification of the computer program sliced the double calculation of damping ratios from the iteration sequence.

3.4 RIGID BEAM EXTENSIONS

The original modified substitute structure method was developed for analysing frames rather than structural walls. In the former case beam and column lengths are considerably greater than the joint dimensions and hence the consideration of the joints as a single point is an acceptable detail of the structure modelling. In structural wall systems this is not the case and failure to include measures to model the joint width will lead to serious errors. If a joint is to be modelled, say at the centre of a fifteen-foot wall, then the joint can be considered as having a width of seven and a half feet before connecting to a typical four-foot long lintel beam. In this manner the width of the beam-wall joint reduces the effective length of the lintel beam and increases its stiffness.

Several solutions are possible to solve this problem, probably the crudest is to include in the model extra members which are rigid and inextensible. These extra members would be rigidly connected at the centre of the wall and at the face of the lintel beam. This would give the true end of the lintel beam the same rotation and lateral displacement as the centreline of the wall. Two problems are inherent in this solution. The first of these is that extra degrees of freedom will be required for the extra joints at the interface of the lintel beam and rigid member. The extra joints will also increase the half-bandwidth of the stiffness matrix. Both these factors increase CPU requirements and the cost of running the program. The second problem is that to use 'rigid' members requires the use of a very large moment of inertia for the cross section of these members. This can tend somewhat to dominate the stiffness matrix and if too large a value is used it can reduce the accuracy of the results. On the other hand, a lower value of the moment of inertia, while being more satisfactory for use in the stiffness matrix, defeats the aims of a rigid member. When the two test structures were analyzed using extra beams and a single precision stiffness matrix it was found that the best compromise for this situation was to use a rigid beam moment of inertia of approximately thirty times that of the wall to which it was connected.

Another possible solution to the problem of non-zero joint size lies in the conception of a new member. This member along with the associated member degrees of freedom is illustrated in figure 3.4. The member displacements can be fully described by the degrees of freedom at the center of the wall. This element has a member stiffness matrix composed of three parts: an axial portion similar to that of a member of length L; a bending portion also similar to that of a member of length L; and an extra stiffness incurred from the rigid ends. This extra stiffness matrix which corresponds to the unprimed degrees of freedom of figure 3.4 is shown in table 3.1. This matrix does not require that the rigid ends of the member be of equal length which allows for the possibility of a coupled structural wall with unequal wall depths. It should also be noted that, as expected, the matrix goes to zero when the member has no rigid extensions and hence reverts back to the case of a frame element.

During the program input for the structure the extensions on each end are read in; if either is non-zero the extra stiffness matrix due to rigid extension is calculated and added to the structure stiffness matrix. After the displacements have been calculated for the joints, the displacements are computed for the ends of the flexible member by using the relationsips between the displacements at each end of the rigid beam. That is, that horizontal displacement and rotation are the same at both ends of the rigid section and the vertical displacement is equal to that of the center wall joint modified by the appropriate addition or subtraction of the product of rigid arm rotation and length. With the end displacements of the flexible region known, the member forces can be calculated in a manner similar to that of any normal member of length L.

At this stage the program is set up to handle rigid extensions on only horizontal members with fixed ends as these are the only ones of concern for structural wall systems.

After inclusion of the provisions for rigid extensions testing was performed to determine the accuracy and effectiveness of the inclusion. A one-story, one-bay test frame was used to remove 'bugs' from the routines. The one-story, one-bay frame offers an excellent means of testing. As there is only one mode and a limited number of members and joints, hand calculations can easily be performed as a check.

The next structure tested was a six-story, one-bay structure with rigid extensions on both ends of the beams. A run was made with this structure using extra members for the rigid beams and another run using the method of adding the rigid extensions by the use of the three segment element already described. Upon convergence the value of all factors of concern was found to be equal to all reasonable significant figures. The run in which extra members were used to model the rigid arms required 39 per cent more degrees of freedom (50 vs. 36) and a 45 per cent increase in the half-bandwidth (13 vs. 9) when compared to the same structure modelled by use of the three segment elements. For this structure the use of the composite member for handling joints of finite width saved 38 per cent of the CPU time requirements (3.91 seconds vs. 6.348 seconds) over using extra 'rigid' members to model the joints.

CHAPTER 4 TESTING THE PROGRAM FOR ELASTIC CAPABILITIES.

The writing of any computer program to solve a given problem is always subject to inaccuracies caused by roundoff error or incorrect logic. Even if the program is written with extreme care, minor errors may creep in that can produce results indicating the solution alogrithm is not valid when the difficulties may lie with the programming of that alogrithm.

The computer program which was developed to apply the modified substitute structure method to shear walls underwent a series of tests to examine its elastic capabilities before being regarded as acceptable. Some of these tests will now be related as they serve to demonstrate some of the practical concerns for a functioning elastic modal analysis program.

4.1 TESTING THE STIFFNESS MATRIX FORMULATION AND EIGENVALUE PRODUCTION.

The first item of concern with the analysis of a program is to ascertain that all input data is being read correctly by the computer. This is achieved through a complete 'echo printing' of the input data before further operations occur. Although this is standard programming practice and not particular to a modal analysis program it is a point too frequently overlooked.

Having established that the input data is correct, the programmer must then check the building of the stiffness matrix. This is accomplished by having the stiffness matrix output in a file where it can be examined separately. In the case of small structures this is performed by doing hand calculations while for larger structures by comparing with stiffness matrices produced from computations performed on an identical structure by a proven static analysis program. A further test that can be performed to check the stiffness matrix production while also checking the eigenvalue routine is through the examination of the frequencies and mode shapes from a simple structure. A pair of very elementary examples for this which will help to pinpoint errors at an early stage are the horizontal and vertical pendulums. These structures though almost trivial in nature provide examples which can be easily confirmed by hand calculation. This is a consideration of noteworthy importance in the choosing of structures to test during the early analysis stage of a computer program. The ability to test structures which can be verified by knowledge of the 'correct' answer avoids the complication of trying to rationalize the differences arising from the use of two independent programs neither of which may be correct. These simple structures are shown in figure 4.1 (a and b). Table 4.1 gives the algebraic expressions for relevant properties such as frequency, displacement and forces to be expected when the the program is operating correctly. These formulas may be verified by realizing that the horizontal pendulum is analogous to the standard cart on frictionless rollers attached to a spring of stiffness constant k as shown in figure 4.1c. In the case of the pendulum though the axial stiffness is determined from the extensional stiffness of the uniform rod.

If the mass is only attached to the horizontal degree of freedom or if the mass is also attached to the vertical degree of freedom but with the pendulum properties chosen so that the bending mode frequency is well separated from that of the axial mode, then the modal participation factor for the horizontal mode should be plus or minus unity. The uncertainty in sign is a result of the fact that while eigenvectors can be normalized on one arbitrarily chosen displacement the magnitude or sign is never known except in relative terms.

If the program is working correctly with axial and bending modes well separated, the axial mode should not produce displacements or forces in the vertical or rotational directions. With the modal participation factor being unity the axial force would correctly be the value of the spectral acceleration multiplied by the horizontal mass. When hand calculations are performed the spectral acceleration is taken from a graphical representation of the spectrum. If the spectral acceleration value is given in terms of a fraction of gravity instead of an absolute acceleration then the spectral acceleration value will have to be multipled by the acceleration of gravity to make the preceding equality true. With the forces in the rod known, the displacements can be easily determined from elementary strength of materials.

In the same manner testing of the vertical pendulum checks the bending mode of the bar. The stiffness in this mode can be equated to that of a cantilever with a point load located at the tip acting perpendicular to the axis of the cantilever. This mode should not produce any axial force in the member though shear should arise as well as a bending moment at the base.

Shear deflections are usually not considered in normal modal analysis as the frames under consideration are usually made up of long slender members for which deflections due to shear are insignificant when compared to those due to bending. With the use of the more stocky members found in structural walls it becomes desirable to include in the program the capability of computing shear deflections. This must also be reflected in the construction of the stiffness matrix before determination of the natural frequencies, since the allowance of shear deflections will make the structure more flexible, resulting in longer periods than would otherwise be the case. The shear deflection provision can be tested during the elastic testing of the program in the same manner as bending deflection. The vertical cantilever again forms a good test structure and the algebraic expressions for the pertinent results are shown in table 4.1. As shear deflection is so frequently ignored in analysis it is desirable that a program having the ability to calculate it also have provisions by which the calculation of shear deflection can be bypassed. This is accomplished in this program by a placing a zero value for either the shear modulus of the structure or of the shear area of members for which the shear deflection is not desired. In this manner shear deflection can be considered in individual members and not in others or by the change of one number in the data file can be totally ignored for the whole structure.

The discussion of the pendulums used for test purposes raised the problem of whether or not masses should be associated with the vertical degrees of freedom as well as those representing horizontal motion. The pendulums are rather specialized test structures and this point is of more interest in the larger structures that are of a more realistic nature. A problem arises because if masses are attached to vertical as well as horizontal degrees of freedom then computation costs can increase by as much as a factor of two. The "vertical" masses create extra mode shapes which may cause an unwanted contribution to the vector sum of forces that are excited by a horizontal spectrum. In the case of one of the test structures examined with masses attached to vertical degrees of freedom, these produced almost pure axial column lengthening as one of the higher modes. Though the horizontal displacements were all very small and of varying sign for this mode the vertical displacements were all in the same direction producing a large modal participation factor for this high numbered mode. If a program is designed to analyse structures for which it is important that vertical inertia forces be included then that program must keep track of which masses are associated with horizontal forces in order that they only have the acceleration from the horizontal spectrum applied to them. When masses are separated according to the direction of motion which they oppose and the appropriate spectral acceleration values are applied accordingly, then structure modes which are primarily vertical will not induce significant forces from the horizontal acceleration.

The necessity of attaching vertical masses to a structure can often be determined from an examination of the amplitudes when only horizontal masses are attached. The amplitude of any degree of freedom in any one mode is give by formula 4.1:

$$X= A \operatorname{Sin}\omega t \qquad (4.1)$$

where A is the maximum amplitude.

If this is differentiated twice then equation 4.2 gives the acceleration of the same point:

$$X=-A\,\omega^2 \operatorname{Sin}\,\omega t. \qquad (4.2)$$

Hence the maximum acceleration of a point on the structure will be $A\omega^2$. For any give mode the value of ω^2 will be the same for all points and hence the acceleration of the nodes will be directly proportional to their displacements. Therefore, an examination of the relative magnitudes of the horizontal and vertical displacements will show if there is a large component of vertical acceleration that should have an inertia force associated with it. Examination of several trial structures that were used in testing the frame analysis program has shown that the vertical acceleration of the column line nodes is in the range of two orders of magnitude lower than the horizontal acceleration. The low proportion of vertical acceleration reflects the large axial stiffness present in the columns relative to their bending stiffness. The correct modal analysis of structures which have masses attached off the column lines could quite easily form the topic for a separate thesis; as this is not a problem in the shear wall structures that are of concern here all future references to masses in this paper will refer solely to masses associated with horizontal inertia forces.

4.2 COMPARISON WITH ANOTHER ELASTIC MODAL ANALYSIS PROGRAM.

Another method of checking the results of a new program is by comparison with results of an existing and previously tested routine. One such program that was available for this purpose was the program 'DYNAMIC'. This program had been written in the early seventies and while its logic and language is somewhat dated in terms of modern programming style it has a variety of options that make it a powerful elastic analysis program which is known to have produced valid results on several occasions. The first tests for comparision of the two programs was performed on a five story frame structure shown in figure 4.2. This structure was the same five story structure tested by Shibata and Sozen[7] and reported in their 1975 paper on the substitute structure method. Results other than the natural periods are not listed in their paper for elastic analysis, but the periods they list agree well with those obtained from the two programs under examination here.

In this structure the results produced by the two programs were very close. Slight differences (mostly in the order of one percent) in the results listed for forces and displacements were attributed to differences in input data. These results are shown in table 4.2. The input data for the two programs varied slightly as one program required input in foot units while the other program required that the data be in inch units. As the input properties are only given to three figures this causes slight differences in the output produced. Another source of difference was in the spectrum used; while the modified substitute structure method program was using a National Building Code spectrum directly, 'DYNAMIC' used a Newmark-Beta spectrum which had been adjusted to represent an NBC spectrum. As both 'DYNAMIC' and the elastic component of the modified substitute structure method program operate completely independently, this agreement was judged to be a valid indication that both programs were able to produce accurate results when testing this size and style of structure.

At this time it is appropriate to discuss the units that go into the makeup of the stiffness matrix. In using the Imperial system the joint coordinates that produce member lengths are frequently input in feet while the member properties are in square inches and inches to the fourth. A common unit of length must be chosen to construct the stiffness matrix. At first examination the choice would seem to be an arbitrary one with the inches being favored as final deflections are perhaps better 'felt' in inches and the use of inches in the stiffness matrix would save the necessity for conversion later. Although the use of inches in the stiffness matrix would be correct the use of a common unit of feet produces a better conditioned stiffness matrix. This is because the terms making up the stiffness matrix do not contain a length factor to a uniform power and the use of the larger length factor tends to equalize the magnitude of terms in the stiffness matrix. While structural properties can be imagined for which this is not true examination of some structures such as the five story structure shown in figure 4.2 shows that foot units do reduce the ratio of largest to smallest elements lying on the stiffness matrix diagonal. For example in the five story frame a ratio of the largest to smallest diagonal element is 527 when inch units are used in the construction of the stiffness matrix but when foot units are used the ratio drops to 4.5. This reinforces the theme that internal use of foot units provides a better conditioned stiffness matrix than internal use of inch units.

The adequate testing of some subroutines may require that they be copied totally from the program into a second program whose sole purpose is the calling of the subroutine under a logical variety of circumstances. This proved to be the case for the subroutine that was used to calculate the spectral acceleration from an input of natural period and damping. Though the standard test runs produced satisfactory answers it was not until very low damping values were tested that it was found that the spectrum routine was in error and corrections could be made. A thorough examination showed that this error occurred only during one of the more rarely summoned logical paths of the subroutine. Under these circumstances the only certain method of checking the subroutine was to use a 'driver' program which logically went through different values of damping and period while calling the spectral acceleration and printing out all three values to be checked by hand. It is only through tedious effort and checking such as this that any sort of real confidence can be developed in the program's ability to produce accurate results.

In many cases the use of double or extended precision will be regarded as an extravagant waste of CPU time to achieve a level of accuracy that is unnecessarily high. In the analysis of a small structure with member stiffnesses approximately equal to each other the use of extended precision is probably not necessary. However in the analysis of large structures and in the cases where through large variations in section properties a wide range of values exists in the stiffness matrix then the extra accuracy is required. One such structure that proved to require double precision was encountered in this testing program and will be referred to as 'structure A' shown in figure 4.3. This structure has several features which did not aid in its analysis. For example it incorporated short, high moment of inertia, 'rigid' beams and the top four members were of a different material and rigidity than the remainder of the structure. Although it did not seem to be the case with 'structure A', it is not difficult to conceive of structures in which a flexible top section acts as a 'free vibration damper' greatly affecting the modal results. This danger becomes acute when a flexible region of a structure has an independent fundamental period which coincides closely to that of one of the lower modes of the whole structure. This is not the case with structure 'A', as can be realized when the top section is separated and analysed as a self contained structure. The eigenvalues produced show the top section to have a frequency placing it a respectable distance from any of the lower periods of the total structure.

Another feature of 'structure A' which makes it difficult to analyse is the presence of the short stubby beams. They were included in the model to represent an offset in a column center-line and had to transfer the resulting moments and downward forces without exhibiting large differences in deflection between their ends. While it is possible to 'juggle' the degrees of freedom in a structure to make the deflections of one point agree with those of another, it is difficult to do so without destroying some of the equilibrium equations for the structure. Although it was tempting to assign the same vertical and rotational degrees of freedom to corresponding ends of the stubby beams, this would have eliminated the corresponding moment caused by the offset of the column line.

In view of the consistency of the results found when testing the five story structure under the two modal programs, there was some considerable surprise and puzzlement when the results of analysing 'structure A' showed the two programs to differ by up to one hundred percent for some of the member forces. At this stage it was not certain which if either program was producing the 'correct' answers and a lengthy search for the cause of the differences resulted.

The first thought was that one of the programs was not dimensioned large enough for the structure. The modified substitute structure method program was checked for this by running on 'Interative Fortran'. This is a Fortran compiler available on the UBC system which performs more extensive error checking than the standard fortran compiler, including checking for dimensioning errors. For this reason it is more expensive to run and is used primarily for the 'debugging' of programs. The modified substitute structure method program passed the Interative Fortran test and as 'DYNAMIC' had analysed structures of far greater size dimensioning was eliminated as a cause of the differences.

It was noted that the first major discrepancies in the analyses appeared in those values printed from the eigenvalue finding routine. Hence interest shifted to the comparison of the information and more importantly the arrays entering this routine. Testing and comparison of stiffness and mass matrices was performed by having the programs modified to print these arrays on sequential files. Other computer programs were then written which used these files as their input data. The first of these auxiliary programs compared the stiffness matrix terms from the two dynamic analysis programs on a term for term basis. As the magnitudes of the terms varies considerably within the matrix this was done by computing a ratio between the elements rather than trying to calculate a numerical difference between any two corresponding terms. Provisions were made in this program to ascertain that zero valued elements corresponded without producing infinite valued ratios during this comparison. The mass matrices were also copied to their own sequential files in the same manner as the stiffness matrices and underwent similar element to element comparisions.

A second auxiliary program provided the opportunity to cheaply test the stiffness matrices in an eigenvalue routine under controlled conditions. This program was written such that the only input given to it was a stiffness matrix and a mass matrix, each in a separate sequential file. This routine produced eigenvalues in a manner which eliminated differences not attributable purely to differences in the matrices entering the eigenvalue finding routine.

The operation of the program consisted mostly of transferring the data from the mass and stiffness sequential files into arrays, calling on the eigenvalue finding routines and printing the resulting eigenvalues. By varying the assignment of the input files it was possible to find the eigenvalues that would be produced when the mass matrix that would be used in one modal analysis program is placed into the routine accompanied by the stiffness matrix from a separate modal analysis program. In this manner the causes of the eigenvalue discrepancies could be uniquely determined. It was through the use of these two auxiliary programs that the necessity of extended precision was appreciated for the correct analysis of 'structure A'

At the time these tests were being performed the modified substitute structure method program had been modified to permit elastic modal analysis but was still operating completely in single precision. However, 'DYNAMIC' constructed its stiffness matrix, computed eigenvalues and vectors and carried out most major options in extended precision.

Using the first auxiliary program it was determined that the difference in the stiffness and mass matrices was very slight. When compared on an element for element basis the members of the stiffness matrix produced by single precision were minus 0.126 percent to plus 0.028 percent different from those produced by double precision. However the eigenvalue for the first mode for these matrices differed by almost ten percent. This difference gradually decreased with increasing mode number and the eigenvalue corresponding to the tenth mode was different by less than a tenth of a percent. Exchanging the mass matrices used in the eigenvalue routines did little to change the eigenvalues produced by each of the stiffness matrices. This led to the conclusion that the difference lay in the use of a single precision routine or a double precision routine to construct the stiffness matrix. This conclusion was verified by further tests after the routine which utilized single precision was converted to double precision by reassigning the stiffness matrix formation arrays and variables used to double precision. Apart from changing single precision real constants and variables to double precision constants, no changes were made in the executable statements in the routine. Once these steps had been implemented the eigenvalues produced were essentially the same as those from the original double precision routine.

Minor differences in the order of one or two percent could now easily be tolerated. These differences were attributed to the different units of input and the variations in acceleration spectrum as described earlier. These differences cannot be regarded as significant due to the experimental errors which are inherent in the physical measuring of the input properties.

After these changes were performed the program that had been produced to perform elastic modal analysis and compute the damage ratios expected by the modified substitute structure method was renamed 'EDAM' . This stands for Elastic and/or Damage Affected Modal analysis and differentiates the program from any others using the method.

4.3 COMPARISON WITH ELASTIC TIME STEP RESULTS.

Perhaps the most rigorous way to check the elastic capabilites of a modal analysis program is to compare the results with those produced by time step analysis using an earthquake which has a spectrum which matches that used in the modal analysis. This method of program examination not only determines if the alogrithm is operating but also ascertains the viability of modal analysis and the appropriateness of the spectrum. To carry out these tests a time step program must be accessable. At least two such options were available at UBC with the program chosen being DRAIN-2D. This program has been developed at the University of California at Berkeley[3] and its use has been reported in several studies involving time step analysis[7,2]. The properties of the program will be discussed more fully in sections of this work dealing with the inelastic testing of the modified substitute structure method program. At this time it is sufficient to state that DRAIN-2D has the capability to compute the force and displacement envelopes for a structure of fairly arbitrary configuration and member properties when undergoing a set of accelerations which are part of the input data. To achieve results unaffected by inelastic action it is noted that DRAIN-2D performs elastic analysis when the yield moment of the members of the structure is not exceeded; this is easily prevented either by specifying a low acceleration or by setting the yield level of the members at a high value.

In order to apply time step analysis tests were undertaken using the program DRAIN-2D and the first ten seconds of four records, these being two components of the Kern County (Taft) 1952 earthquake and two components of the El Centro 1940 event. The accelerations were specified for this earthquake at intervals corresponding to 0.02 Seconds and using a linear interpolation between acceleration points, a time step interval corresponding to 100 hertz was used.

In testing the program EDAM against DRAIN-2D elastic runs were performed on a five story frame, this being Shibata and Sozen's five story frame modified by the arbitrary addition of 9 foot rigid arms on the beam ends. In the modal analysis Spectrum 'A' from Shibata and Sozen[7] was used as it is an appropriate spectrum for the records chosen. This spectrum is shown in Figure 5.7. Five percent damping was used in both DRAIN-2D and modal analysis runs. For the purpose of comparison, the largest bending moment for each member was examined. As 'spectrum A' is an average spectrum for the earthquake records used, the results of the four DRAIN-2D runs were averaged before comparing with those of modal analysis. These results are presented in figure 4.4. The results can only be described as excellent, with the spectrum moments all being within 6 percent of those predicted by the average of the time step runs. They form an almost text book example of the viability of the modal-spectrum approach to elastic analysis. It should be noted that not only were the computation cost for modal analysis an order of magnitude below those of the time step analysis but the data file preparation for the modal analysis was considerably easier and less time consuming.

It was thus through testing several structures of varying size, complexity and features on two completely independent modal analysis programs and a time step program that it was established that the program under examination could truly produce valid results for elastic modal analysis. This is an important step in determining that the program can produce valid inelastic results by a modification of the elastic method.

CHAPTER 5 TESTING THE INELASTIC PREDICTIONS OF THE METHOD.

With the elastic capability of the program established, we are in a position to assess the accuracy of the method with respect to inelastic behavior. However, this is not as simple as the test for errors in the elastic range. While elastic modal analysis is a well established practice, the use of the modified substitute structure method to predict inelastic actions is treading on much newer ground.

The viability of the method was assessed in two ways. The first was to examine the trends in ductility demand predicted by the program, comparing these to those trends reported by other researchers in the literature. The second approach was to examine several test structures which could be compared on a numerical basis with results obtained from the inelastic time step analysis program DRAIN-2D.

5.1 Literature Comparison of Damage Patterns

Many researchers have shown that the ductility demand on the coupling beams of wall systems is highest in the area of one-third the distance up the height of the structure. This caused concern during the first attempts to compare damage patterns from the modified substitute structure method with those of published papers. All the initial test structures that were modelled, although apparently reasonable in their properties, showed the heaviest damage ratios to occur in the coupling beams at the top of the walls. Causes for this discrepency are related below.

The reason that the maximum ductility demand occurs below the top of the structure, as found by other researchers, lies in the dual method of lateral load carrying by the wall. The lateral force imparts a flexural deflection to the walls which, if the lintels had a low moment capacity, would put the largest damage ratios for the structure in the top lintel. This effect is offset, however, because when the shear in the lintel causes axial forces in the walls, the resulting axial deformations relieve some of the flexural stress in the coupling beams. The effect is much more dramatic towards the top of the structure as the axial wall deformations are cumulative from the base.

When tests were performed on the program EDAM involving a sixteen-story structure with wall and beam section properties similar to the eighteen-story building which Paulay[4] had analysed by the laminar method, it was found that the maximum coupling beam damage ratios predicted by the program occurred in the range of one-third to one-half of the height of the structure, confirming Paulay's predictions. To examine the effect that this axial shortening of the walls has on the damage ratios, another run was performed in which the wall area was multiplied by a factor of ten while all other structural details were held constant. As expected, the axial deformations of the walls were reduced by one order of magnitude, and the center of major coupling beam damage shifted towards the top of the structure. The axial deformations decrease the damage ratios of the coupling beams in the structure and, in this case, the larger area walls led to damage ratios 30 percent greater than those of the original run.

The shifting of the largest damage ratio downward from the top of the structure can only be expected to occur when the axial deformations of the walls are significant relative to the displacements of the coupling beam ends. Hence it will be less pronounced in structures with wide walls, as this tends to increase the displacement of the lintel beam ends. It will also be less pronounced when the lintels are more flexible or have lower yield moments, since each of these reduces the shear in the lintel and hence the axial force and deformation caused in the walls. Finally, the effect will be less prominent in those structures which have walls with a high ratio of cross-sectional area to cross-sectional moment of inertia.

The structure that was tested by Paulay was modelled from an elevator or stair shaft wall and was composed of two channels connected by coupling beams. Compared to the structures with simple planar walls tested here, this structure had a much lower ratio of area to moment of inertia, and narrower joints (modelled by shorter rigid arms). It should be noted that Paulay's structure shows up one of the failings of the modified substitute structure method as presently formulated: it is only truly applicable to members with symmetric sections. This is a result of the assumption that all members will have the same ultimate moment regardless of which side is in compression. In a channel section, this assumption is not valid, as the moment capacity is as unsymmetrical as the concrete distribution about the neutral axis. The analysis of such a coupled channel section by the method will only be valid if any inelastic behavior is restricted to the coupling beams.

These comments on the influence of axial deformations on nonlinear behavior, point to one of the problems that would be encountered with 'lumping' walls of a structure to reduce computation costs. While it may often be possible to combine walls that are exactly similar by multiplying the structural properties and loads of the first wall by the number of similar walls, this procedure may lead to difficulties with dissimilar walls. If the two walls that are 'lumped' together have differences in stiffness properties, then the damage ratios so determined will be incorrect.

Another point for practical consideration is that during the lateral analysis of a structure it is common to ignore the columns, although they are awarded a fixed proportion of the vertical load. While this might be a valid assumption where the wall is undergoing small vertical deformations, the columns would interact to carry different vertical loads if the vertical deformations of the walls should get too large.

As the Paulay structure shows damage patterns quite typical of the findings of other researchers, it was concluded after examining this structure that the modified substitute structure method was capable of reproducing the general damage patterns correctly.

5.2 Assumptions for Comparison with a Time Step Analysis Program

After establishing that the pattern prediction was reasonable it was then necessary to examine numerical predictions of inelastic behavior by comparison with time-step results. The requirements and choice of a time step analysis program that is viable for the analysis of structural walls is governed by both material, structural and geometrical considerations. Analysis of structural walls constructed of concrete requires a hysteresis loop that is appropriate for that material. This consideration eliminates many finite element programs which, while satisfactory in all other respects, consider a concrete member only in terms of pure elastoplasticity, not differentiating between loading and unloading stiffness curves or other items which are important in the post yield analysis of concrete members.

It is also necessary that any method used to check the assumptions af another method do so in a manner that takes a thorough account of the factors most likely to influence the results. In the analysis of coupled walls it is important that there be no restricting assumptions concerning the location of inflection points in the members, since these points will be very differently located in the coupling beams and in the walls. In the fundamental response mode, the walls will act like two cantilevers with a large base moment. Thus, most, if not all, the wall segments will have no inflection points, whereas in the coupling beams, there will generally be a central point of infection.

Another structural factor worthy of consideration is the interation between the axial load and the yield moment of the walls. The walls will be subjected to alternating tensile and compressive loads; while the latter will increase the moment capacity of the wall, tensile loads may lower the capacity to the point where yielding occurs. Thus the nodes should be allowed three degrees of freedom to permit axial deformation of the walls, to show the reduced ductility demands on the coupling beams, and reflect the axial force imparted to the walls and consequent change in yield moment.

It is these conderations, as well as the desire to use a reputable time step analysis program, which led to the choice of DRAIN-2D. Through other studies[7], including experimental work, the program has been demonstrated to have the capacity to handle structural walls and to produce reasonable results. DRAIN-2D was written at the University of California at Berkeley[3]. The program uses a step-by-step dynamic analysis procedure in which an acceleration, specified as part of the input data, acts upon a structure of arbitrary configuration. The program handles the degradation of concrete stiffness with the use of an extended version of Takeda's model, and is capable of reflecting the effect of axial force on the yield moment of concrete sections.

Test structures were chosen to test the method in a variety of situations covering a comprehensive range of the relevant parameters, while attempting to reduce the structures tested to a reasonable number. The structures were modelled by a set of line members connected by joints located at each floor level to which the members were rigidly connected at each end. Hence each wall is broken into a number of segments equal to the number of stories in the structure. The joints describing the location of the walls were placed on the neutral axis of the uncracked section. Structural properties used in the test structures were based on member sizes and properties approximating those used in practice. Thus, member sections with reasonable material properties, steel quantities and locations were analysed to get the member properties. Although the area and initial moment of inertia of the walls were usually held constant throughout the height of the structure, the yield moment of the wall was assumed to vary linearly throughout the height of the structure. This was to reflect the fact that the moment capacity of a wall increases with increasing axial load toward the base of the structure. This latter point turned out to be somewhat academic for the structures tested, since when hinging occurred in the walls it always took place in the bottom story. For the structures used in this study, the reduction of the moment capacity because of decreasing dead load at greater heights in the wall always had a much smaller effect than the reduction in the applied moment as a function of height.

The ultimate moment-axial force distribution for the member was obtained on the basis of standard concrete section analysis. A linear strain relationship was used with a maximum compressive strain of 0.003 in the concrete. The Whitney stress block with ACI code provisions, was used to compute the contribution of the concrete to the capacity of the section. Consistent with these provisions, no strength was given to the concrete in tension. The steel, assumed to be placed in discrete layers, was modelled as perfectly elastic-perfectly plastic in both tension and compression. The layers of steel frequently regarded as 'temperature steel' were included in these analyses as their large lever arms produce a sizable contribution to the moment capacity of the member. Should the engineer view this as an esoteric exercise applicable only to the researcher with access to large computer funds, it is worthwhile commenting that the calculation of the ultimate moment-axial curves for sections with up to 19 layers of steel were all performed on a programmable pocket calculator. A typical curve take approximatly one-half hour to calculate and plot, including the input of section data.

The walls were connected by a series of coupling beams, whose sectional properties and capacity were kept constant throughout the wall height. The coupling beams were modelled as a member with three sections, a deformable central region equal in length to the clearspan of the member, and two rigid ends stretching from the face of the wall to its center-line. The method of including this member in the modified substitute structure program has been described in section 3.4. It might well occur in practice that during the resistance of the seismic forces, the neutral axis of the wall shifts away from its location in the uncracked wall. This would have the effect of changing the length of the rigid arms and the resulting forces applied to the coupling beams, usually in an unconservative manner. This point seems to be ignored in time step analysis of structural walls and the effect is also not considered in the modified substitute structure analysis. Shear deflection was not included in the calculation of member forces and displacements. The validity of this assumption will be demonstrated in an example later in this chapter.

To determine the masses that should be applied it was assumed that the walls were spaced at fifty feet normal to their own plane and that the load on each floor was 150 lb/sq ft., this being a combination of dead and live load. Each wall was assumed to have a tributary area equal to its length plus half the span of the lintel beam times fifty feet. It was also assumed that while the structure could be imagined as having columns taking up about fifty percent of this load in the vertical direction, the horizontal mass should comprise the total load on the tributary area. Due to the greater stiffness of the walls they would act to take the horizontal force long before the columns took any horizontal load. The vertical force on the walls was used only to determine the ultimate moment capacity of the wall from its ultimate moment-axial curve when using the program EDAM and no vertical forces were placed on the structure during dynamic runs. In the program DRAIN-2D, the capability exists to reproduce the moment-axial curve for the member and to place static preloads on the structure before the dynamic analysis begins. Hence, the vertical forces which had been used in calculating the ultimate capacity of members for the program EDAM were placed as predefined static loads for the time step analysis.

Initial tests of the program EDAM had shown that, with the exception of axial forces in the lintels, the results were similar regardless of whether one or two masses per floor were attached as long as the total mass was kept constant. Also, computation costs increased as the number of attached masses increased. Therefore, it was decided that only one mass per floor should be assumed. When using the DRAIN-2D analysis however, two masses per floor were attached, partly to see if the axial forces generated in the lintels were as low as expected, and partly to check that these forces could be assumed to have a negligible effect on the moment capacities. In practice any axial force in the lintel beams would be partially dissipated in the floor slabs which, though weak in flexure, provide a good axial connection.

The dynamic analysis of reinforced concrete structures frequently provokes debate on the appropriate member properties. In the analysis for static loads, gross moments of inertia and cross-sectional areas are frequently used, partly because the results will be little affected by other refinements as long as all members are treated consistently, but also because better estimates are often not available in the analysis stages. In the analysis of dynamic loads this assumption cannot be made so lightly. If the cracked moment of inertia is used instead of the gross moment of inertia, then the flexibility will be affected and hence, the period and dynamic loads acting on the structure. Shibata and Sozen's[8] original development of the Substitute Structure method took this into account by proposing that the gross moment of inertia be used, but that cracking be accounted for by dividing by 2 if axial compression is present or otherwise by 3. This scheme was used in the work on the modified substitute structure method performed by Yoshida[10].

The assigning of stiffness values in the use of the program DRAIN-2D is not as simple a procedure. The manual for the program suggests using the flexural rigidity value for the cracked section, though it notes that "considerable experience and experimentation will be needed before the element properties can be specified with confidence"[3]. The use of the cracked section is important since the hysteresis rules employed in the program DRAIN-2D use the same section modulus up to first yielding; using the gross section modulus throughout this range would clearly involve too large a stiffness. It was decided to base the cracked section modulus on the same assumptions that were used in the Modified Substitute Structure method. This insures that unyielded members have the same properties in both analyses. In any case, more detailed approximations of the cracked section modulus are are usually beyond the scope of a design method.

For axial stiffness, the total section area was input in both cases because, although cracking would be expected to decrease the section modulus, it would not be expected to affect response to a compressive axial load significantly. Were the member to be in tension, it would be expected that the gross area would be too great, but neither DRAIN-2D nor the program EDAM reduce the areas of members to take account of cracking, and results for structures with concrete members in tension should be viewed with caution.

It was always assumed that the structures tested were rigidly connected to an unyielding foundation. This would represent the commoner case where shear walls terminate in more massive basement walls.

It is not necessary to choose a value of damping for the modified substitute structure method as damping is determined during execution. However, it is necessary to determine such a value for use with the program DRAIN-2D, and 2 percent of critical was chosen as reflecting normal elastic damping; it was included as stiffness proportional damping. The energy lost in hysteretic damping by a structure undergoing inelastic action is automatically accounted for with the program DRAIN-2D.

It was necessary to choose an appropriate set of earthquakes for the time step analysis and a matching spectrum for the modified substitute structure analysis. This was resolved by using the same earthquakes and spectrum that had been used in the early examination of the substitute structure method by Shibata and Sozen[8]. The earthquakes used, including appropriate details, may be found in table 5.1. The records were scaled linearly to give a peak acceleration equal to the desired maximum ground acceleration for the structure. The spectrum used in the modified substitute structure method was spectrum 'A' which had been developed by Shibata and Sozen[8].

Most of the structures tested were of a form shown in figure 5.1 and represent a single pair of coupled walls. Relevant structure properties of these walls can be found in table 5.2.

In comparing ductility values it is vital to ascertain that a similar definition is used for this term in all cases. A logical definition used for both time step analysis and the modified substitute structure method is that of member ductility. This can be defined with respect to the angle between the tangent to the member at its end and the chord joining the ends of the member: it is the value of this angle at response divided by the value at first yield. The measurement of this angle along with a more familiar view of it from a paper by Paulay[6] is shown in figure 5.2. This is equivalent to the term 'damage ratio' used in the modified substitute structure method. Although this can be measured at two ends of any member under both positive and negative moment giving four possible values of ductility, some of which may be equal, the largest ductility demand determined is the one of concern and the one that is used in the comparisons that follow.

5.3 Results and Comparisons with Time Step Programs

(a) Five Story Structural Wall

The first inelastic test structures consisted of three sets of five story structural walls, used to examine the applicability of the method to small structural walls. Two values of coupling beam capacity, 60 Kip-Ft and 100 Kip-Ft were tested at a maximum ground acceleration of 20 percent of gravity. The higher beam capacity was also tested at a ground acceleration of 50 percent of gravity. Although changing the capacity of the lintel beams and maximum acceleration altered the amount of inelasticty in the structures, none of the changes altered the initial elastic period of the structure. The results of these tests were all very similar--the modified substitute structure method predicted correctly the pattern of ductility requirements and deflections but was very conservative, predicting values 50 to 100 percent greater than DRAIN-2D runs. The results for 'series B' tests on the five story wall (which used 100 Kip-Ft lintels and 20 percent gravity) are shown in figure 5.3 and 5.4 for ductility and deflection.

The five-story wall examined in the original tests was very stiff and with the mass used, had a fundamental period of only 0.22 seconds. For a given damping, Shibata and Sozen's spectrum 'A' is constant between 0.15 and 0.4 seconds so that any softening of structures falling in this period range will not result in a lowering of the spectral acceleration response. As noted in chapter 2 this contravenes one of the restrictions on the substitute structure method. To examine the effect of increasing the fundamental undamaged period to more than 0.4 seconds, the original mass used in the five-story wall analysis was multiplied by a factor of 4. This changed the fundamental period to 0.45 seconds. The structure with this revised mass was then analysed by both the modified substitute structure method and by DRAIN-2D. Results in terms of deflection and ductility demand are shown in figures 5.5 and 5.6; they are considerably more encouraging as they indicate results for the modified substitute structure method more akin to the average of the four time step results.

From these results it was concluded that the modified substitute structure method, while giving qualitatively correct damage and deflection patterns, may give results that are numerically conservative when the acceleration response does not decrease with period. Figure 5.7 shows spectrum 'A' along with fundamental periods of the structures examined in this study. The undamaged fundamental period should be greater than 0.4 seconds for accurate results to be produced with this spectrum.

(b) Ten-story wall

The next set of tests was performed on a ten-story coupled wall. Figure 5.9 shows the deflection results for these tests while figure 5.10 shows graphically the ductility demand of the coupling beams. Although the results of the tests show the modified substitute structure method provides a conservative values for the records used, both deflection and ductility estimates are very reasonable. While the modified substitute structure method predicts a deflection for the structure of 3.75 inches the deflection envelopes produced from the DRAIN-2D computer runs indicate a top deflection of 2.5 to 3.5 inches. In terms of ductility demand, the modified substitute structure method predicts the largest coupling beam damage ratio to be 7.55 while DRAIN-2D runs indicate that it lies between 5.05 and 7.25. When uncertainties of the structure and earthquake parameters are considered the results for this ten-story wall are very encouraging.

(c) Sixteen-story wall with an extra uncoupled wall.

The next tests were performed on a sixteen-story wall which had been previously reported by Fintel and Gosh[2]. The initial results for this wall are shown in figure 5.10 which shows the ductility demand of the coupling beams estimated with four different sets of structural parameters using the program DRAIN-2D with the first 10 seconds of the El Centro East-West record.

The first of these is curve 'A' which corresponds to the ductility demand estimated by Fintel for the largest possible earthquake for the structure. Although these results were obtained from the University of British Columbia version of DRAIN-2D they agree well with those results published by Fintel. These results correspond to damping, exclusive of hysteretic damping, of ten percent. However, it was our feeling that non-hysteretic damping, representing the effect of non-structural components, should be less than this since all the structural damping would be reflected in the hysteretic effects. In a program such as DRAIN-2D any inelastic action will result in hysteretic damping and it is not necessary to duplicate this by extra stiffness proportional damping.

Curve 'B' of figure 5.10 shows the ductility demand of the coupling beams when the stiffness proportional damping is lowered to 2 percent. This has a considerable effect on the damage experienced in the coupling beams with maximum ductility demands rising from 9.8 to 17.5. At this value of ductility demand the 5 percent strain hardening ratio on the coupling beams causes them to reach a moment almost twice their original capacity. Hence, run 'C' was performed in which the strain hardening ratio was dropped to 0.5 percent thus placing it closer to the elastic-perfectly plastic idealization. As strain softening rather than strain hardening may occur, especially at high ductility demands, the use of a very low value of strain hardening is an appropiate assumption. Curve 'C' shows the results that are obtained using 0.5 percent strain hardening and 2 percent stiffness proportional damping. Note, of course that the differences between the analysis and that of Fintel and Ghosh do not result from the methods, but simply from the choice of structural parameters. The damping values used here correspond with the smeared damping values proposed by Shibata and Sozen, but the latter can easily be changed in the modified substitute structure method to agree that those of Fintel and Ghosh if desired. Similarly, if strain hardening is felt to be approprite that can be input to the modified substitute structure method.

Curve 'D' was performed to confirm the contention that shear deflections need not be included in structural wall analysis. In run 'D' no shear deflections were included, producing results almost indistinguishable from run 'C' in which the shear deflections have been included. This also reflects that the predominant behavior of structural walls is flexural rather than shear.

Figures 5.11 and 5.12 respectively show results of deflections and displacements for four earthquakes when run on DRAIN-2D and compared to the results predicted by the modified substitute structure method. The deflection estimates for this structure are very consistent for all DRAIN-2D runs and the modified substitute structure method. While the latter method predicts a top deflection 2.88 inches, the time step runs place this deflection between 2.82 and 3.28 inches. The estimates of ductility demand show much greater scatter with values having a range of eight. Figure 5.13 shows graphically the average of the four DRAIN-2D results and modified substitute structure method. Both in terms of distribution and numerical agreement, the modified substitute structure method gives an excellent estimate of the average of four time step runs.

It should be noted that although these tests indicate damage ratios for which it may not be possible to design, the purpose of these tests is to examine the ability of the modified substitute structure method to estimate the results that would be obtained from time step inelastic analysis given that the same assumptions are used in each analysis. The results of the tests on the sixteen-story wall demonstrate that even with large ductility demands, the method is capable of reproducing time step results. This sixteen-story structure forms a good test as it contains many attributes which might give the modified substitute structure method difficulty: the walls have a stiffness change at midheight, the mass is not constant throughout the height of the structure and hinging occurs in the base of the walls.

Examining the results presented in this chapter produces at least two observations worth noting. Without calculating the spectrum for a series of individual earthquakes, it is not possible to predict which of a series of records will produce the most dramatic effect on a given structure. For example the El Centro East-West record produces the largest deflections and ductility demands for the ten and five-story walls but the Kern S69E record shows the largest values for the sixteen-story wall. The results also show that ductility demand has a much greater scatter when different records are examined than does deflection and attempts to determine ductility demands to three significant figures is a futile effort.

5.4 Costs of Execution

As a final item of concern, computing costs should be examined to determine the economic viability of the method. Figure 5.14 shows the costs of a single run for various sized structures on elastic modal analysis, the modified substitute structure method and DRAIN-2D. In all cases the charges include the cost of printing the input data and sufficient output for evaluation of the results. Also the structures represented on this figure are all single pairs of coupled walls conneced by lintel beams at each floor. The graph shows costs for normal priority batch jobs in a not-for-profit computing center, and the figures are only representative of relative costs. Commercial charges could be at least four times the costs shown in figure 5.14. Savings with the modified substitute structure over the DRAIN-2D analysis are only indicative of the cost of a single run; they increase significantly if it is decided to test the structure with more than one earthquake record. For runs using a specific earthquake or series of earthquakes using a program such as DRAIN-2D, it is necessary first to determine the frequencies of the structure for calculation of the damping parameters. Even under these circumstances, where it has been firmly decided to use a program such as DRAIN-2D, it would be worthwhile to run a program such as EDAM which in addition to determining the intial periods of the structure give the designer an excellent indication of the ductility demands to be expected.

CHAPTER 6 APPLICATION OF THE METHOD THROUGH A DESIGN EXAMPLE

6.1 Analysis for the design of a sixteen-story structural wall.

Having examined the applicability and limitations of the modified substitute structure method it is now appropriate to demonstrate how it can be used in a hypothetical design. The example chosen is a sixteen-story structural wall, of a typical height for residential or office buildings using this system for lateral force resistance. In this example, the maximum lateral design acceleration for the site is given as 0.3 times that of gravity with the spectrum of the 1940 El Centro.

The assumptions concerning floor loading, section properties, and other such details are similar to those discussed in section 5.2 for the structures that underwent inelastic testing. These assumptions should not be regarded as necessary restrictions, but simply as a basis by which reasonable values can be chosen. For example, the use of an input floor load of 150 lb/ft^2 would obviously be the designer's choice. Its selection in this analysis should have no effect on the validity of the method. The building under consideration has structural walls of a symmetric design as shown in figure 6.1.

These walls must be designed to carry the lateral load of the structure.

The first step in applying the method is to determine that the structure under examination satisfies the necessary restrictions. In the case of the sixteen-story structural wall in the example, this is a fairly simple procedure. The wall is considered a component of a residential building without flanges on wall ends, so the element is symmetric. Thus, no difficulties will be encountered as would have occurred if the wall had a greater capacity in the positive horizontal direction than in the opposing direction. The system is to be analyzed as a plane frame structure and is of such a nature that torsion is not a problem. As the walls are continuous to the ground, no abrupt changes in mass or stiffness are apparent over their height. Using light partitioning walls, or isolating those walls which might interfere with response and are not considered in the model of the building, the structure meets the criterion of non-interference of non-structural elements. We assume that all the joints and elements will be reinforced as necessary for ductility; in fact, the main purpose of this analysis is to determine the ductility demands so that proper design can prevent catastrophic failure. From this brief examination it is determined that the wall is one that can be analyzed by the modified substitute structure method.

Having decided that the building meets the restriction criteria for the method, it is now necessary to model the structure. It is at this stage that the designer uses his judgement to make assumptions regarding such factors as the values of cracked moment of inertia and horizontal mass. As the lateral force analysis usually follows that of the vertical force resistance and architectural layout, the gross size of members and the locations of joint centers would already have been determined.

The next step is coding of the structure and an initial run of the program. During this procedure the designer will appreciate the virtues of data generators which can be applied to the structure type he most frequently encounters. For example most of the structures used in this study were modelled as two walls and their connecting coupling beams. A data generator which can easily produce a data file for a structure with two column lines was used. However, data generators for reasons of generality have not been included in the program and in this study were written and used separately. It may be the case that a structure under consideration by the designer cannot be modelled by only one coupled wall but must be modelled by a larger set of walls connected by inextensible hinged links which represent the effect of a floor diaphragm. Such a case was illustrated in the sixteen-story test structure of Chapter 5.

With the input data generated, which in this case takes up about one hundred lines, an initial run can be made. The damage ratio results of the first run are shown in figure 6.2, while pertinant results such as frequency are shown in Table 6.1. Here, damage at the base of the walls and in the upper lintels is deemed to be unacceptable for the design earthquake, and changes in some of the structure properties are necessary to realize a reduction in damage.

The first change is to increase the moment capacity of the lintels. A doubling of this value is made before the execution of the second run, here an increase from 40 to 80 Kip-Ft. The damage ratios with these increased capacity lintels are shown in figure 6.3. The changes made before the start of this run cause a considerable reduction in the damage ratios of the coupling beams as well as a slight reduction in the damage ratio at the base of the walls. This is to be expected as the coupling action of the walls is increased by a strengthening of the coupling beam.

A third test of the structure was performed to examine the effect of increasing the value of Young's modulus on the damage ratios of the structure. The value typical of 5 Ksi concrete chosen for this run replaces the value representing 4 Ksi concrete used in earlier runs. This change has the effect of increasing the modulus from 3600 Ksi to 4030 Ksi. Although the use of increased concrete strength would also alter the capacity of the members somewhat, this was ignored and no change was made to the capacity or geometrical properties of the members from the previous test. Other tests in the series are performed to examine the effects of changing member strengths on the response of the structure. It is up to the designer to determine if he needs increased concrete strengths to achieve the desired member capacities. The 12 percent increase in the modulus resulted in a 6 percent decrease in the root-mean-square displacement at the top of the structure but also a 4 to 12 percent increase in the damage ratios of the members. An increase in Young's modulus will have a similar effect on the flexural rigidity value for the coupling beams. This stiffening has the effect of attracting larger loads and hence more inelastic action which produces higher damage ratios. Under these circumstances it is probably not worthwhile to pay for increased concrete strength solely to increase the value of Young's modulus to achieve a decrease in the deflections, as the result is minimal. All further tests that are performed on this structure will use a value of Young's modulus that corresponds to that of 4 Ksi concrete.

The designer may wish to reduce the inelastic action in the walls to the point that they avoid any excursions past their yield value and therefore have damage ratios below unity. Such a decision would be consistent with the belief that inelasticity in columns is undesirable as it often occurs in a less ductile manner than when the inelasticty is concentrated only in members without axial load. Hence, the fourth test was performed following the calculation of a new moment-axial curve for the walls with the same cross-section used in the previous tests but with increased steel content. Based on our assumption of the cracked moments of inertia being dependant only on the gross size and presence or absence of axial load, this change in steel area will have no effect on the moment of inertia used and will only influence the moment capacity of the walls. For the initial run with this new wall member the coupling beams were given the reduced capacity of 40 kip-ft to make the run comparable to the first test. Compared with that test, the resulting damage ratios at the base of the walls have now been reduced to below unity (see figure 6.5), but the lintel beams now incur much higher damage ratios. The is due in part to the damping in test number

4 being lower than in the first test, reflecting the lower damage encountered by the major members. The results of test number 4 when compared with test number 1 also show that reducing the damage ratios of the walls may not reduce the displacements. Indeed in this case, they show a 22 percent increase.

The fifth test corresponds to the second test, as in both cases the lintel capacity is doubled from the previous run. With the exception of increasing the lintel capacity, the input for this run was otherwise unchanged from the fourth test. This run, as did the second test, showed clearly the dramatic effect of increasing the lintel beam capacity in reducing the damage ratios, both in those members and in the walls (see figure 6.6). Although the values obtained are possibly within our ability to design in terms of ductility requirements, further tests were needed to reduce the higher damage ratios and to examine some of the properties of this sixteen-story wall.

The sixth test was performed after examining the maximum allowable shear capacity of the lintel using the provisions of the ACI code but ignoring the component of shear carried by the concrete. From the analysis it was found that the lintels could approach a moment capacity of 300 kip-ft without first failing in shear. This value was then used for the ultimate moment capacity of the lintel beams. The run showed that even with this member strength some damage had to be expected in the coupling beams (see figure 6.7). The results also showed that with so high a lintel capacity one of the walls would be in, or very close to, a state of tension. This was viewed as being undesirable for these reinforced concrete elements and hence a seventh run was performed, lowering the lintel capacity to the point where a reduction of only 50% of the vertical load would occur in a wall. Computations of the capacity of the coupling beams to satisfy this criterion can be performed by hand: since the beams are going to be very close to if not at, their yield level, the shear carried by the beam is calculated by dividing twice the moment capacity of the beam by its length. The axial forces in the walls are increased or reduced by the accumulated total of these beam shears, and the appropriate values required to cause a specified reduction in the axial force due to vertical loads are easily computed. A rough idea of the desirable capacity of the coupling beams is thus determined.

The seventh and final run was performed with the value of the moment capacity of the lintels reduced to 130 Kip-Ft for the reasons outlined in the previous paragraph. The results are quite acceptable for all variables examined. The largest damage ratio, as shown in figure 6.8, is 4.7, a figure easily withstood by proper detailing. The largest deflection when compared with the height of the structure at that point is 1/190. The axial forces in the walls induced by the earthquake are 45 percent of the static axial load carried by those members so they are safely away from a state of tension. The damage ratios in the base of the wall are 0.64 which, in terms of economy of section, is probably too low. By examination of sections with the same cross-sectional shape but different steel areas, a steel quantity and distribution can be chosen which places the wall closer to yield in a more economical manner. It should be noted that consistent with the desire to avoid hinges in column members this capacity should be that of the base of the wall under minimum expected axial force.

The seventh run completes our analysis of the the coupled wall as far as the modified substitute structure method is concerned. The final stages of the design involve ensuring that members are detailed to provide sufficient ductility to sustain the damage ratios predicted for the structure.

As a check of the results predicted by the modified substitute structure method for this example (structure number 7), computer runs were performed using the program DRAIN-2D. These runs were made using 2 percent of critical damping and the first ten seconds of the same four earthquake records outlined in Chapter 5. The results of these runs in terms of ductility requirements of the coupling beams are shown in figure 6.9 with deflection estimates shown in figure 6.10. The results show that the modified substitute structure method is a good predictor of both damage ratio and deflection. As the spectrum is an average for the four records used and not an envelope, some ductility demands and deflections from the program DRAIN-2D are greater than those predicted by the modified substitute structure method. Indeed, for this example the results predicted by the method are a very reasonable estimate of the average of the results from the four inelastic time step runs. Excluding the cost of the run necessary to establish the frequency for input of damping to DRAIN-2D the cost of executing the four runs is over twenty times the cost of the single run of the modified substitute structure method. As the spectrum method requires only input of maximum acceleration and spectrum type rather than an extensive string of accelerations, and as the program EDAM outputs damage ratios directly, both input data preparation and program output interpretation are considerably easier when using the modified substitute structure method.

6.2 Examination of the effect of changing maximum ground acceleration.

One of the many advantages of a technique such as the modified substitute structure method is that parametric studies can be performed quickly and cheaply. An example of this is an examination of the effect that maximum ground acceleration changes have on the various structural response parameters. Knowledge of the behavior of the structure under accelerations which differ from the design maximum may be of interest when it is considered how uncertain this maximum is, and a series of tests was performed on a sixteen-story structure similar to the final one obtained in section 6.1. The lintel beams were made six foot long rather than eight foot, and there was a corresponding two foot decrease in the wall centerline spacing. There were no other changes to input data. Damping, calculated by the program, naturally increases at higher accelerations as the members undergo more damage. Hence, it is necessary only to change the maximum acceleration figure in the data file before performing a test run from this series.

In doing this analysis the use of the cracked moment of inertia might appear to render the results invalid for those structures where some members were being stressed insufficiently to cause cracking. To examine this situation the sixteen-story frame used in these acceleration parameter studies was recoded, assigning uncracked moments of inertia to all members having a damage ratio equal to or less than 0.25. These results were then compared with a run in which cracked sections had been used throughout. In recoding it had been necessary to change the moment of inertia values for the five top stories of the walls, and changes in fundamental frequency, damage ratios and displacements were in the order of 1 percent and were therefore judged to be insignificant. On the basis of these results, using the cracked section for an entire structure appears to be an acceptable procedure. This process should, however, be re-examined if the damage ratios in the bottom story are low enough to suggest that cracking has not occurred in this region.

The individual results of these tests will not be reproduced here, but figures 6.11 to 6.13 show the trends. Figure 6.11 shows the damage ratios in the coupling beams at three values of maximum acceleration. As expected, the damage ratios were higher at increased values of acceleration, but what is also apparent in this figure, is that the location of highest damage moves up the structure as the acceleration increases. This can be explained with reference to the dual load paths present in coupled structural walls: in the lowest acceleration as shown in figure 6.11, with the lowest ground acceleration, all but three of the coupling beams have already yielded and are carrying the maximum shear. Hence, the maximum axial deformation of the walls is present, giving maximum relief to the damage in the top coupling beams. Higher ground accelerations decrease the relative importance of the effect as the axial strains remain almost constant while wall bending increases.

Figure 6.12 shows the effect on the first two periods of increasing the maximum acceleration. This figure reflects a finding reported in the literature from many shaking table and free vibration tests of damaged reinforced concrete structures. This observation is that the period increases as higher values of acceleration cause more damage and a loss in stiffness. This figure shows the fundamental period to be much more affected than that of the second mode. The trend continues to higher modes, so that by the tenth mode the difference between the damaged and elastic periods is indistinguishable for this sixteen-story wall undergoing a maximum acceleration of fifty percent of gravity.

The reason that the fundamental mode is more affected is that it is more dependant on the stiffness of the first floor walls. If hinges were to form higher up the building, the higher modes would be more affected.

Figure 6.13 shows the effect of increasing ground acceleration on the value of the smeared damping calculated for the first three modes. The graph shows an increase in damping for the fundamental mode with increasing acceleration in the ten to thirty percent of gravity range. Above this range of ground motion, damping is somewhat constant in the 5.5% of critical range. For this structure increased values of excitation have little effect on the damping of the second and third mode.

Figure 6.14 illustrates the effect on the horizontal displacements of the structure of increasing the input ground acceleration. The insert graph shows that despite the non-linear behavior of the coupling beams and, eventually, the formation of hinges at the base of the walls, the final top deflection is almost linear with increasing acceleration. It can also be seen that the deflection is caused mostly by curvature in the lower regions of the walls as higher segments show little curvature.

These last tests are a simple example of how the method can be used to determine the effects on response parameters of changes in a single input variable. For these tests, most changes require only minor editing of the data file to modify the input from one run to the next. The computation and output costs for a typical run are in the order of \$3.50 for a run performed on normal priority on a non-profit basis on the University of British Columbia computing system. Thus the modified substitute structure method is demonstrated to be an economical and practical approach to parametric studies of the seismic response of coupled structural walls.

CHAPTER 7 CONCLUSIONS

The modified substitute structure method has been presented as a design aid for the seismic design of coupled structural walls. The method extends the elastic modal analysis technique into the inelastic range and has been shown to provide good estimates of the ductility requirements and deflections of coupled structural walls resisting lateral forces which place some of the members into their inelastic range.

The coupled structural walls tested in this study were of height ranging from 5 to 16 stories. The method has been shown to give good results in all cases except where the fundamental period of the structure places it on a constant portion of the input spectrum. The accuracy of the results, as determined by comparison with inelastic time step analysis, appears to improve as the fundamantal period of the structure increases.

The method is inexpensive to use and can be performed with a computer program using a data file having only minor changes from that used in static analysis. It is therefore a method that could be used in the practical design of seismic resistant coupled structural walls.

$$
\mathbf{k} = \frac{12EI}{L^4} \cdot
\begin{bmatrix}
0 & & & & & \\
0 & 0 & & \text{(Symmetric)} & & \\
-L1*Y & L1*X & L1*L*(L+L1) & & & \\
0 & 0 & L1*Y & 0 & & \\
0 & 0 & -L1*X & 0 & 0 & \\
-L2*Y & L2*X & \frac{L1+L2}{2} & L2*Y & -L2*X & L^2*L2+L*L2^2
\end{bmatrix}
$$

X= Horizontal projection of member
Y= Vertical projection of member
L= Length of elastic portion of member.
L1= Length of Rigid arm at lesser joint end
L2= Length of Rigid arm at greater joint end.

Table 3.1: Additional Member Stiffness Matrix to Account for Rigid Arms.

	HORIZONTAL PENDULUM	VERTICAL PENDULUM	
		No Shear Deflection	With Shear Deflection
Period	$2\pi\sqrt{\frac{mL}{AE}}$	$2\pi\sqrt{\frac{mL^3}{3EI}}$	$2\pi\sqrt{m\left[\frac{L^3}{3EI} + \frac{L}{A_vG}\right]}$
FORCES			
Axial	$(S_a)(m)(1)$	0	0
Shear	0	$(S_a)(m)(1)$	$(S_a)(m)(1)$
Free End Bending Moment	0	0	0
Fixed End Bending Moment	0	$(S_a)(m)(1)(L)$	$(S_a)(m)(1)(L)$
DISPLACEMENTS (free end)			
Horizontal	$\frac{(S_a)(m)(1)(L)}{AE}$	$\frac{(S_a)(m)(1)L^3}{3EI}$	$\frac{(S_a)(m)(1)(L^3)}{3EI} + \frac{(S_a)(m)(1)(L)}{A_vG}$
Vertical	0	0	
Rotation	0	$\frac{(S_a)(m)(1)L^2}{2EI}$	$\frac{(S_a)(m)(1)L^2}{2EI}$

Table 4.1

Analytic results of Vertical and Horizontal pendulums.

(Free end permitted 3 degrees of freedom but mass only opposes horizontal motion)

Period and Participation factors

Mode	Elastic Periods EDAM	DYNAMIC	Shibata and Sozen	Participation Factor	S_a
1	0.858	0.858	0.85	1.286	0.254
2	0.262	0.262	0.26	0.452	0.545
3	0.137	0.137	0.14	0.253	0.500
4	0.088	0.088	0.087	0.211	0.321
5	0.067	0.067	0.065	−0.111	0.245

ROOT MEAN SQUARE FORCES

MN	AXIAL (KIPS)	SHEAR (KIPS)	BML (K-FT)	BMG (K-FT)
1	14.929	9.449	113.309	113.472
2	9.449	14.926	54.429	113.309
3	9.449	14.961	54.739	113.472
4	11.325	16.853	202.164	202.313
5	26.229	24.004	108.358	160.067
6	26.229	24.032	108.598	160.218
7	11.163	23.308	279.633	279.749
8	49.014	30.551	153.003	186.991
9	49.014	30.573	153.175	187.131
10	10.843	27.420	328.989	329.093
11	75.272	35.841	207.414	188.921
12	75.272	35.861	207.525	189.084
13	7.183	25.317	303.748	303.869
14	99.229	38.972	328.082	102.058
15	99.229	39.025	328.494	102.267

NOTE: Entire mass for each floor is attached to right column
Lesser joint end for beams is left end.
Lesser joint end for columns is lower end.
0.2 times gravity, 5% Damping, First 5 modes used.

Table 4.2

Elastic Modal Analysis results for Shibata and Sozen's 5-Story structure (Figure 4.2) using their Spectrum 'A'

EARTHQUAKE	DATE	AMAX	RECORDING STATION
El Centro (NS)	May 18, 1940	0.348	El Centro site Imperial Valley Irrigation District
El Centro (EW)	May 18, 1940	0.182	El Centro site Imperial Valley Irrigation District
Kern County (S69E)	July 21, 1952	0.179	Taft Lincoln School Tunnel
Kern County (N21E)	July 21, 1952	0.156	Taft Lincoln School Tunnel

NOTE: AMAX = Maximum Acceleration of original record during segment of record used.

First ten seconds of each record used.

Table 5.1

Earthquake records used in DRAIN-2D computer runs.

	5-Story Wall (Series B)	5-Story Wall (Mass*4)	10-Story Wall	16-Story (Design)	16-Story Wall with extra uncoupled wall. Floor 0-8	Floor 9-16
Fundamental Period (Sec.)	.2248	.4496	.8346	1.3485	.8538	
2% Damping Factor	.00143	.00286	.00531	.00858	.00554	
Weight/Floor (Kip)	270	1080	270	270	1050 1475 (top floor)	
Young's Modulus (Ksi)	3600.	3600.	3600.	3600.	3600.	
Structure Height (Ft)	41.75	41.75	84.25	135.25	146.93	
Maximum Ground Acceleration	0.2g	0.2g	0.2g	0.3g	.2271g	
Lintel						
Capacity (Kip-Ft)	100	100	60	130	375	
Clearspan (Ft)	3.5	3.5	6.0	8.0	3.594	
Moment of Inertia (In^4)	1024	1024	1024	1296	34444	
Area (in^2)	144	144	144	144	487.8	
Left coupled wall						
Moment of Inertia (In^4)	2187000	2187000	2187000	5184000	49560000	40470000
Area (in^2)	1620	1620	1620	2160	6308	5150
Rigid Arm (Ft)	7.5	7.5	7.5	10.0	12.8	12.80
Right Coupled Wall						
Moment of Inertia (In^4)	2187000	2187000	2187000	5184000	13290000	10850000
Area (in^2)	1620	1620	1620	2160	4067	3319
Rigid Arm (Ft)	7.5	7.5	7.5	10.[illegible]	8.26	8.26
Uncoupled Wall						
Moment of Inertia (In^4)	-	-	-	-	9547000	9547000
Area (in^2)	-	-	-	-	No Vertical degrees of Freedom on Uncoupled wall.	

Table 5.2

Properties of Test Structures

	Run #1	Run #2	Run #3	Run #4	Run #5	Run #6	Run #7
Maximum Acceleration	0.3g	0.3g	0.3g	0.3g	0.3g	0.3g	0.3g
Lintel Capacity (Kip-Ft)	40	80	80	40	80	300	130
Wall Base Capacity (Kip-Ft)	16600	16600	16600	34711	34711	34711	34711
Wall Top Capacity (Kip-Ft)	14400	14400	14400	27850	27850	27850	27850
Young's Modulus (Ksi)	3600	3600	4030	3600	3600	3600	3600
Results							
Period Mode (Damaged) (1)	2.333	2.068	1.999	2.035	1.913	1.479	1.775
(2)	0.375	0.360	0.344	0.340	0.335	0.311	0.329
(3)	0.135	0.132	0.126	0.126	0.126	0.123	0.125
(4)	0.072	0.072	0.068	0.069	0.069	0.068	0.069
(5)	0.047	0.047	0.044	0.046	0.045	0.045	0.045
Damping Mode (1)	0.072	0.069	0.072	0.035	0.045	0.037	0.049
(2)	0.041	0.040	0.041	0.023	0.025	0.026	0.027
(3)	0.032	0.031	0.032	0.021	0.022	0.022	0.022
(4)	0.028	0.027	0.028	0.020	0.021	0.021	0.021
(5)	0.026	0.025	0.026	0.020	0.020	0.021	0.021
Maximum RMS Displacement (Inches)	9.34	8.50	7.97	11.40	9.68	7.99	8.58
Number of Iterations	11	7	8	5	5	4	3
Spectral Acceleration							
Mode (1)	0.117	0.135	0.136	0.187	0.180	0.252	0.186
(2)	0.891	0.903	0.889	1.085	1.053	1.048	1.030
(3)	0.884	0.872	0.824	0.934	0.920	0.894	0.908
Participation Factor							
Mode (1)	1.49	1.49	1.49	1.50	1.50	1.48	1.49
(2)	-0.73	-0.73	-0.73	-0.74	-0.74	-0.72	-0.74
(3)	0.37	0.38	0.38	0.39	0.39	0.39	0.39
RMS Axial Force at Base (Kips)	108.8				306.7	1070	485

Table 6.1

Results of Computer Runs on 16-Story Design Example

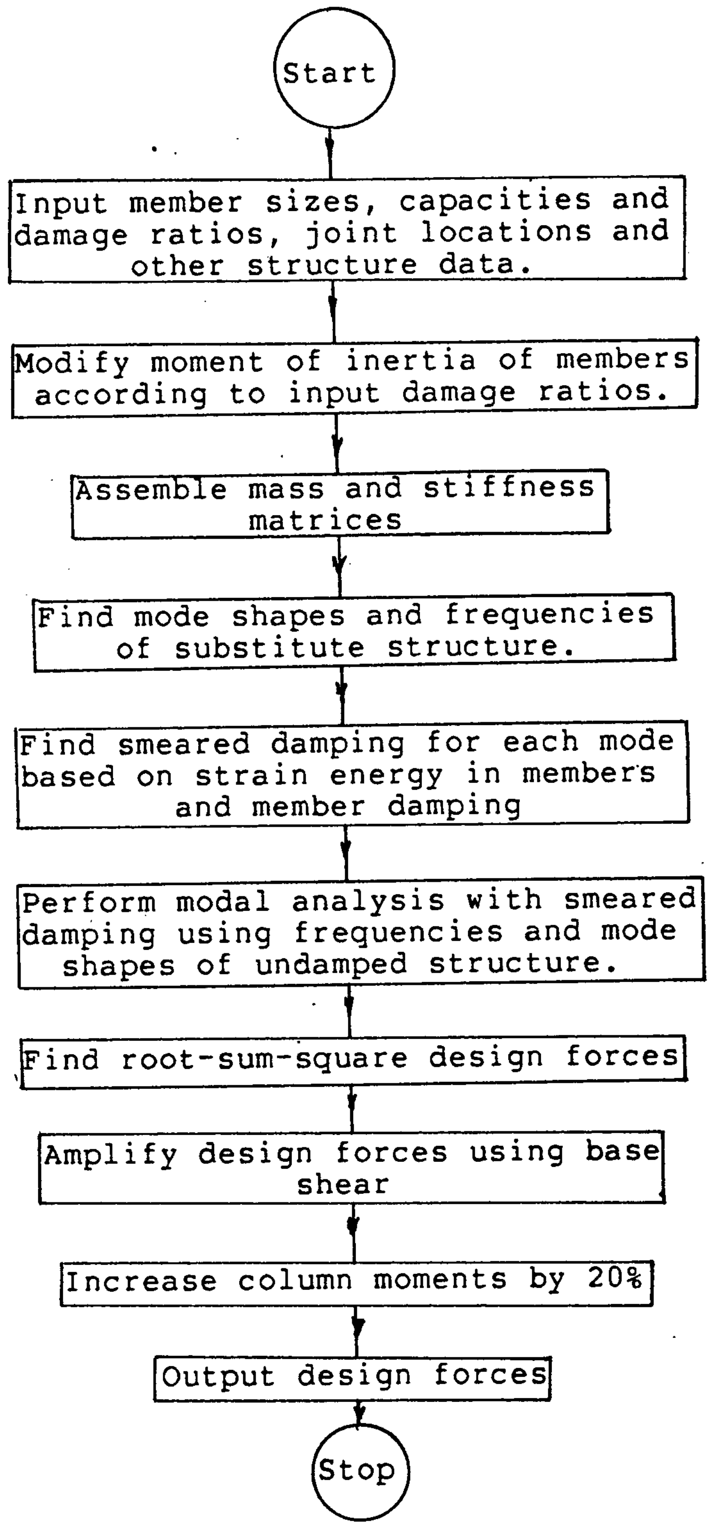


Figure 2.1: Flowchart for the Substitute Structure Method.

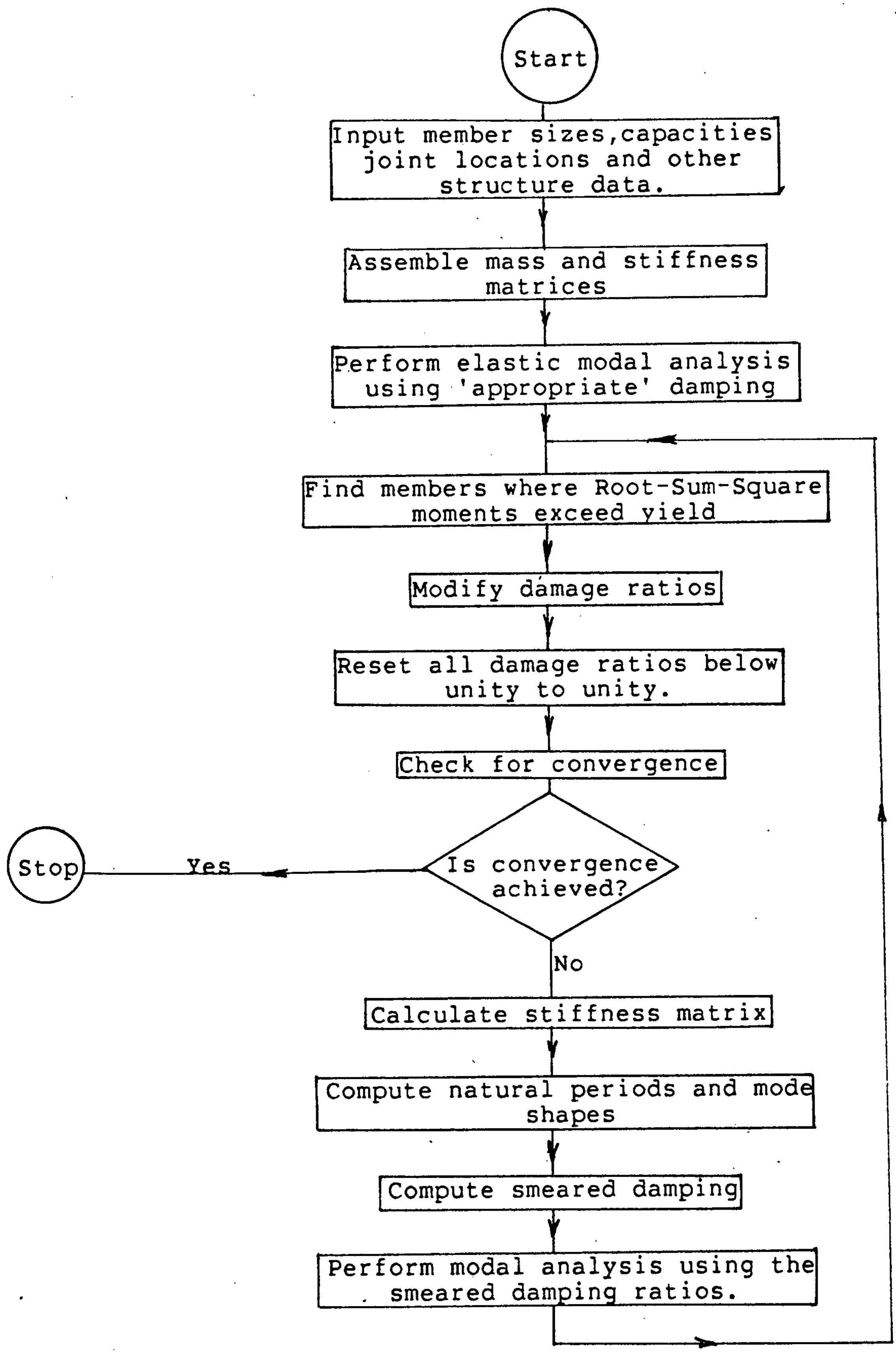
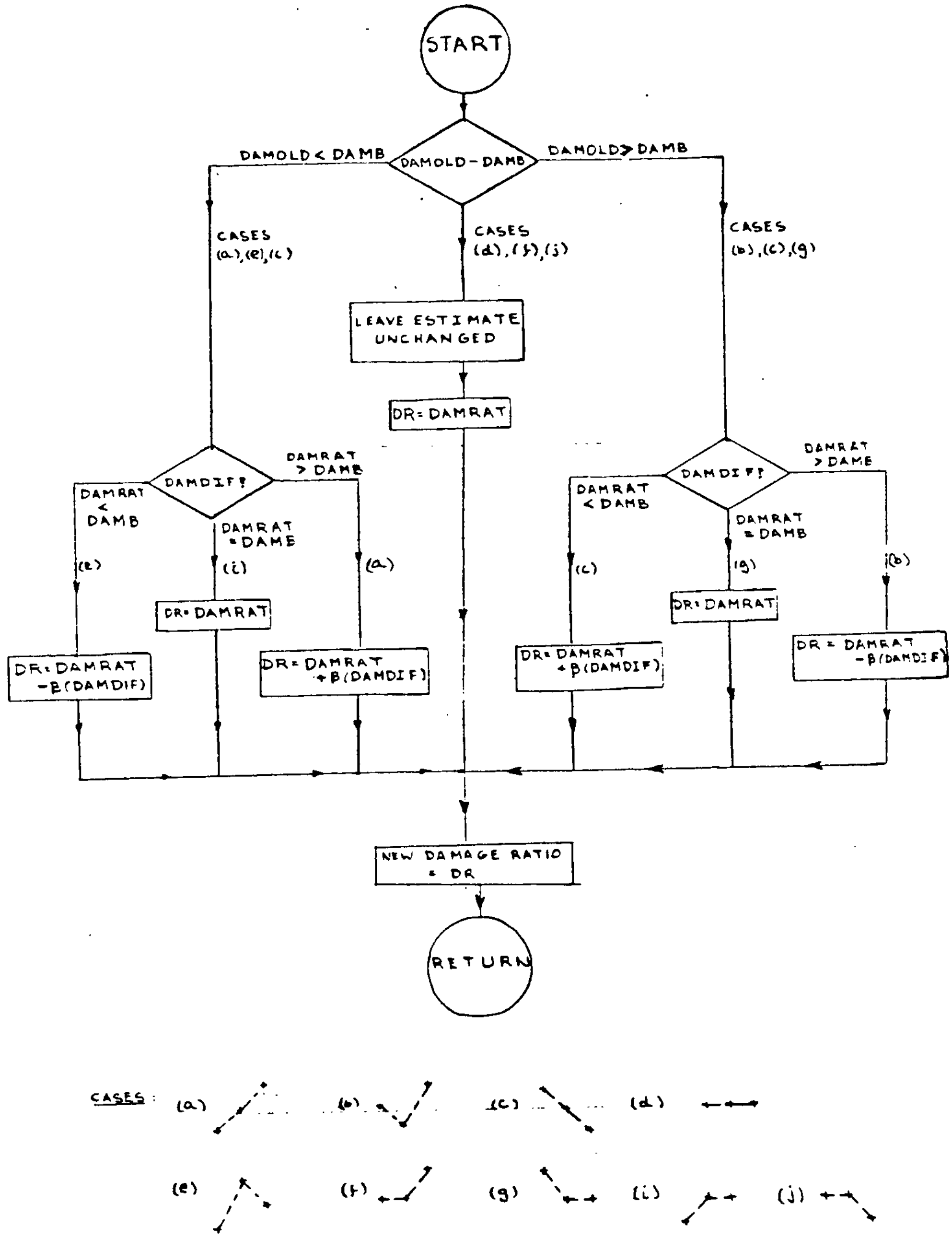


Figure 2.2: Flowchart for the Modified Substitute Structure Method



DAMOLD = Damage Ratio for i-2 Iteration

DAMB = Damage Ratio for i-1 Iteration

DAMRAT = Damage Ratio for i Iteration

DAMDIF = DAMRAT-DAMB

DR = Damage Ratio returned to program.

Figure 3.1: Flowchart for convergence speeding routine

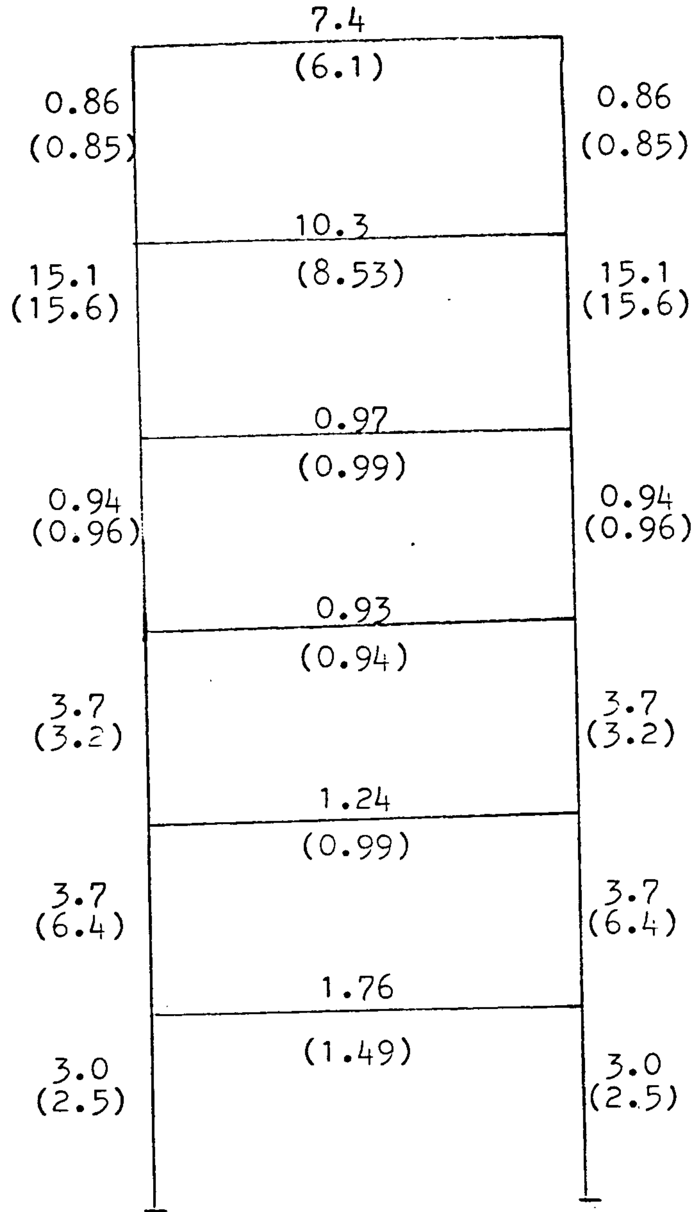


Figure 3.2: Comparison of Damage Ratios using different Convergence schemes on one-bay, six-story frame. (Results of old scheme shown in parenthesis).

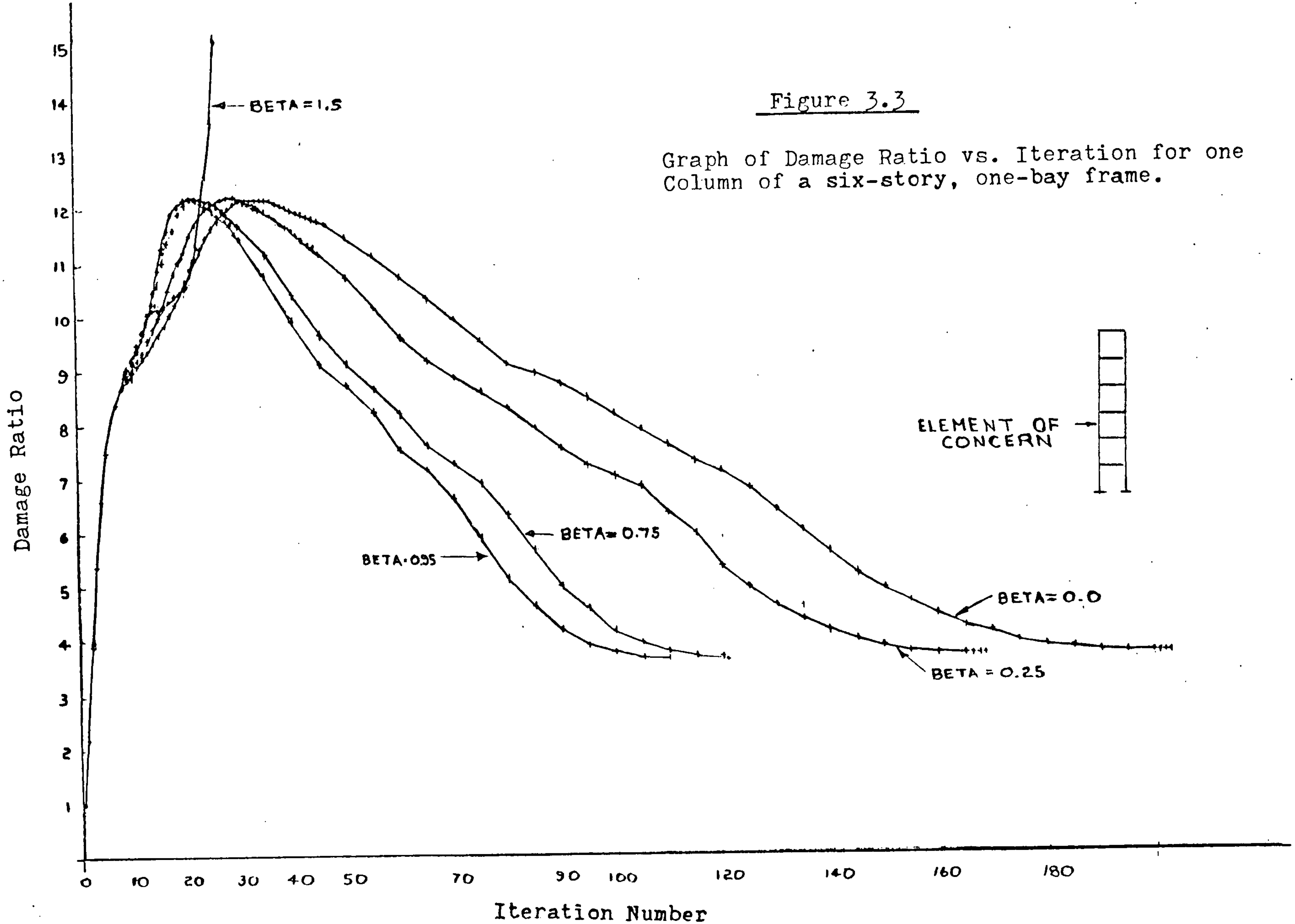


Figure 3.3

Graph of Damage Ratio vs. Iteration for one Column of a six-story, one-bay frame.

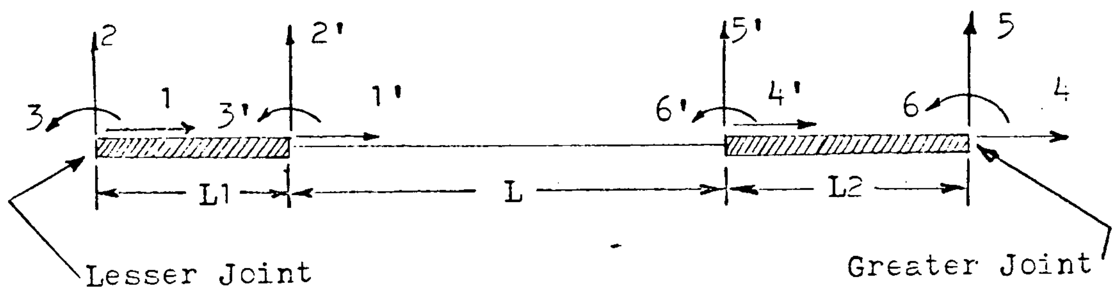


Figure 3.4: Diagram of Modified Member to Include Rigid Extensions.

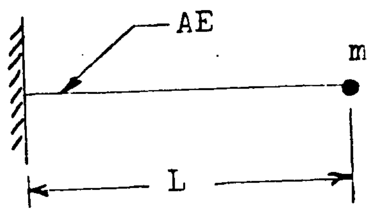


Figure 4.1(a)

Horizontal Pendulum

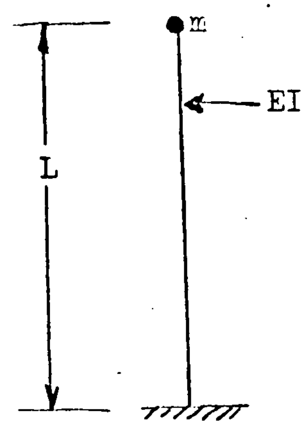


Figure 4.1(b)

Vertical Pendulum

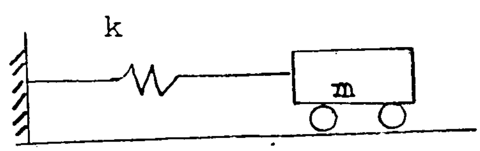


Figure 4.1(c)

Cart on frictionless rollers

Beams: 18" x 30" I=13,300 in^4

Columns: 24" x 24" I=13,824 in^4

Young's Modulus= 3600 Ksi.

Floor Weight = 72 Kips per floor.

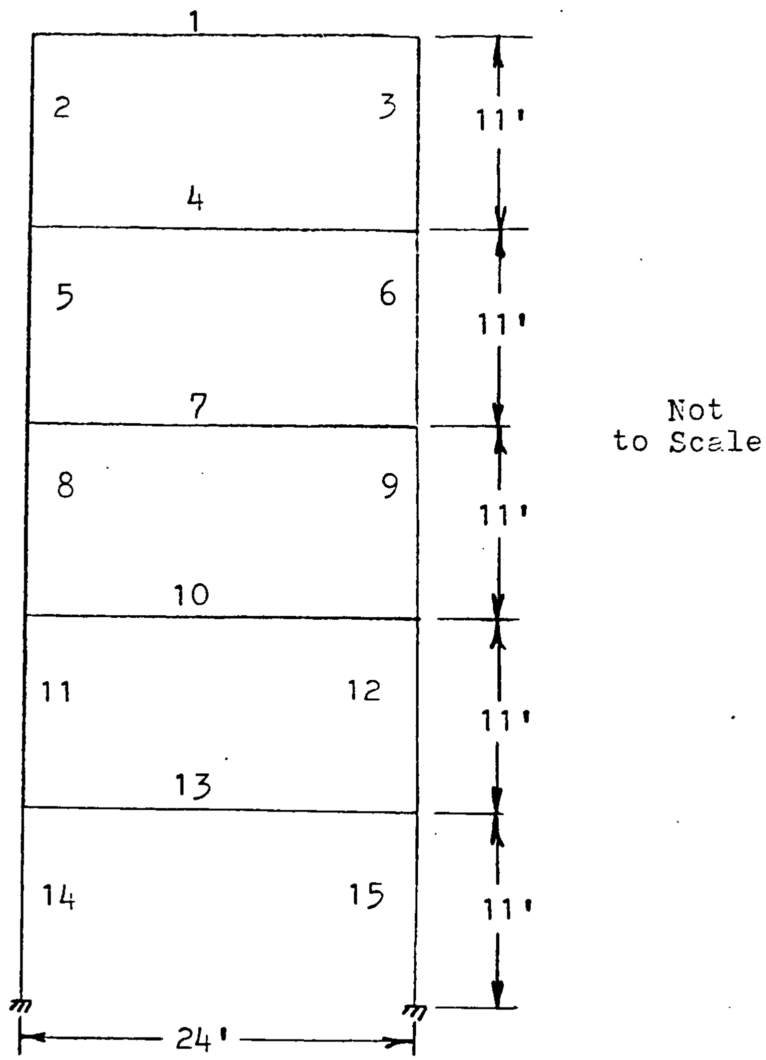


Figure 4.2

Shibata and Sozen's five-story structure

(showing member numbering used to designate Root-Sum-Square forces in tables)

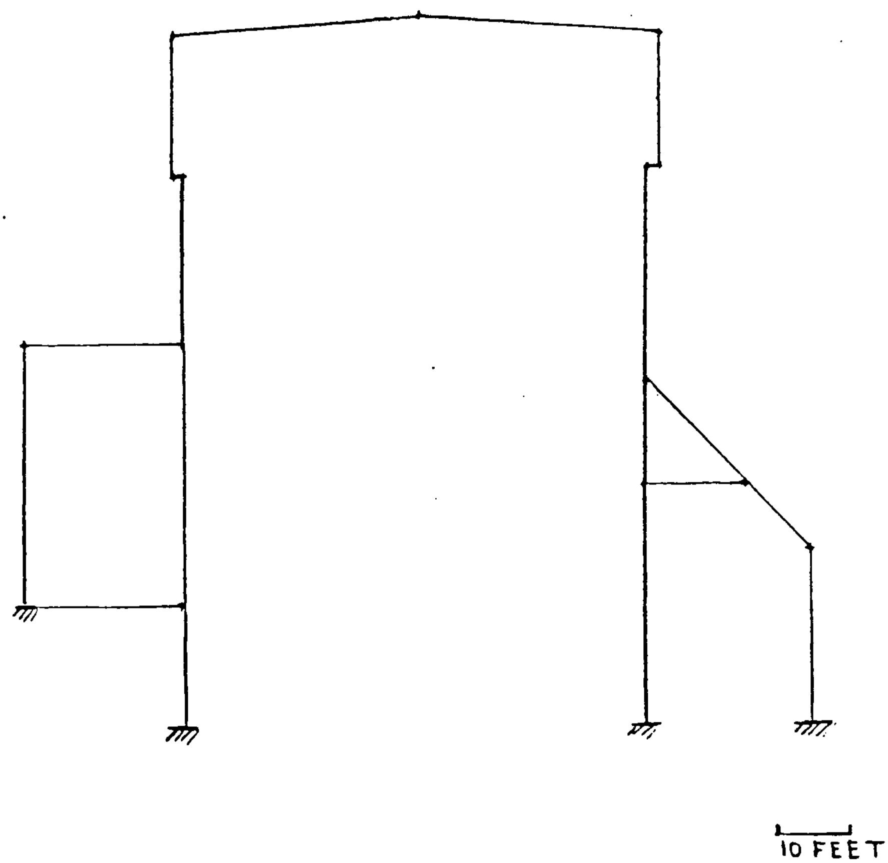
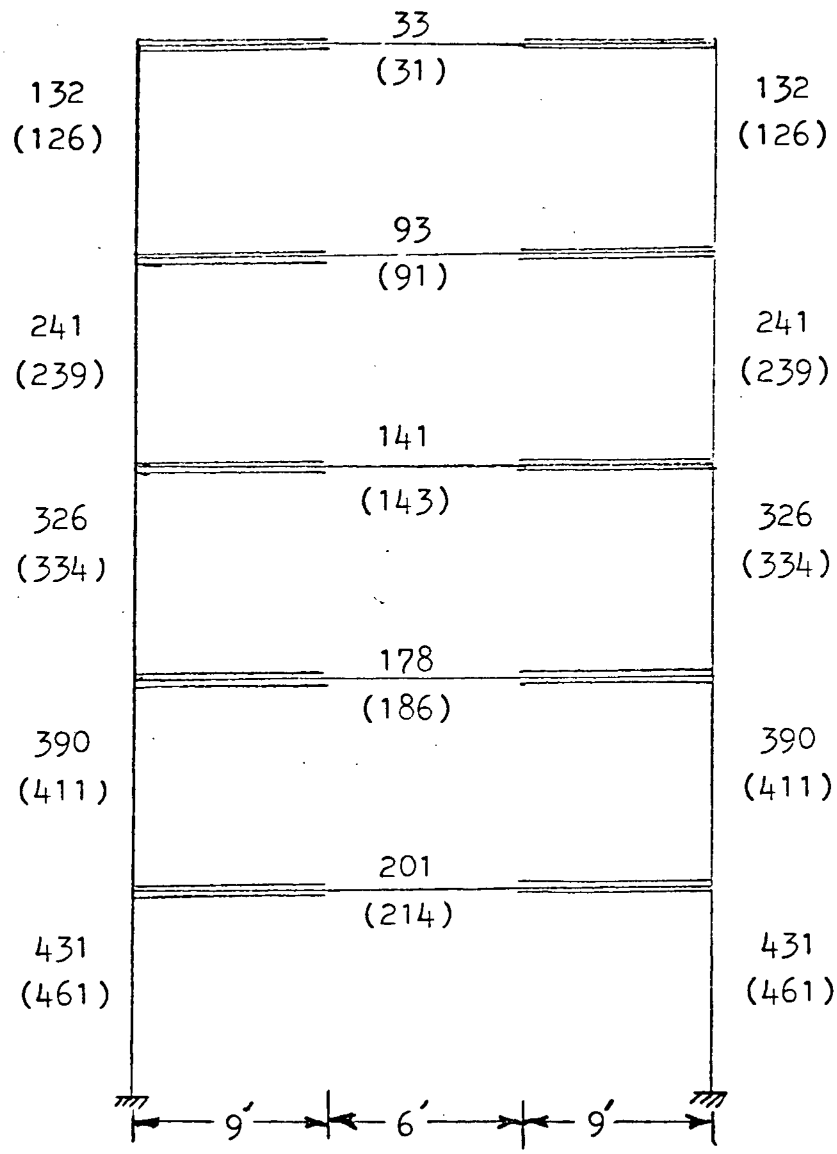


Figure 4.3
Configuration of Test Structure 'A'



Note: Acceleration = 0.2 times gravity.
All Bending moments in Kip-Ft.
DRAIN-2D results shown in paranthesis.

Figure 4.4

Five-Story Structure with rigid arms showing bending moments produced from elastic modal and elastic time step analysis.

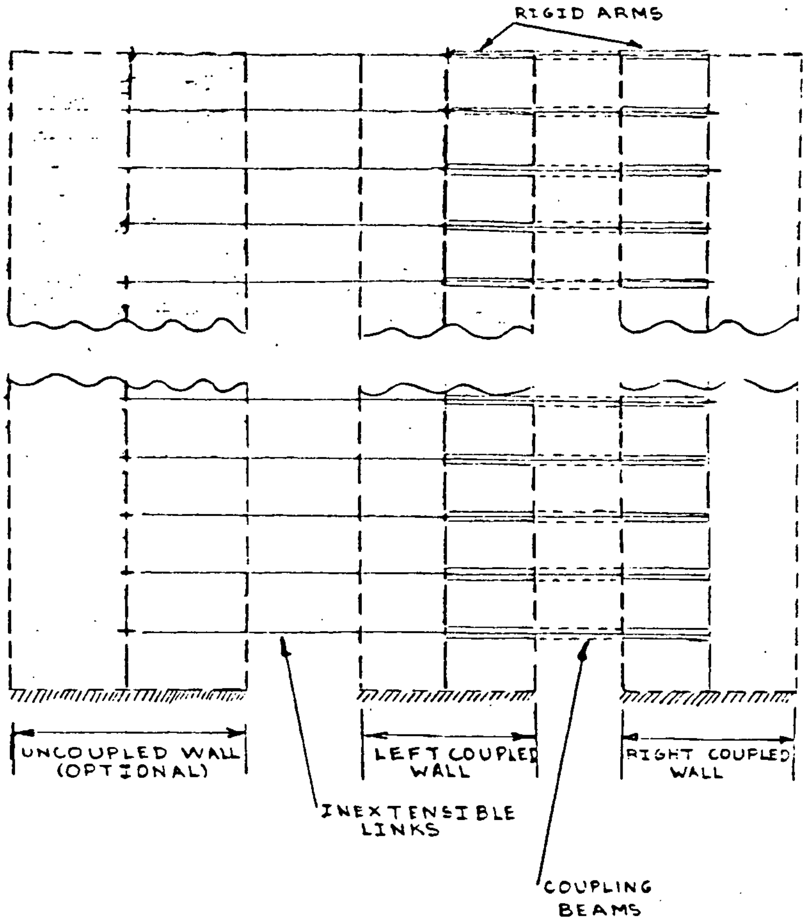


Figure 5.1
General Test Structure Configuration

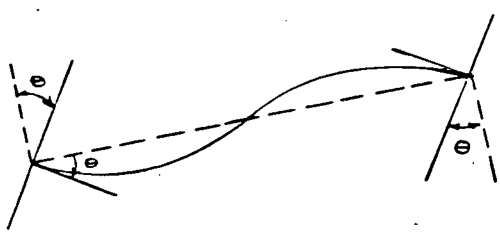


Figure 5.2a
Angle Used For Calculation Of Member Ductility

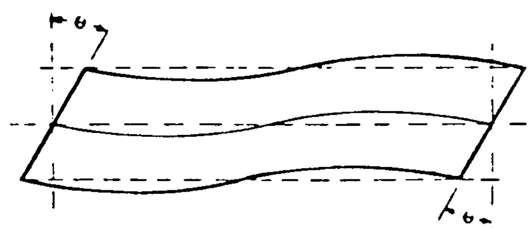


Figure 5.2b
Member Ductility As Given By Paulay (Figure From Ref. 6)

$$\mu = \frac{\theta_{ULT}}{\theta_{YIELD}}$$

Figure 5.3
Ductility Demand Of the Coupling Beams for the 5-Story Wall (Test Series 'B')

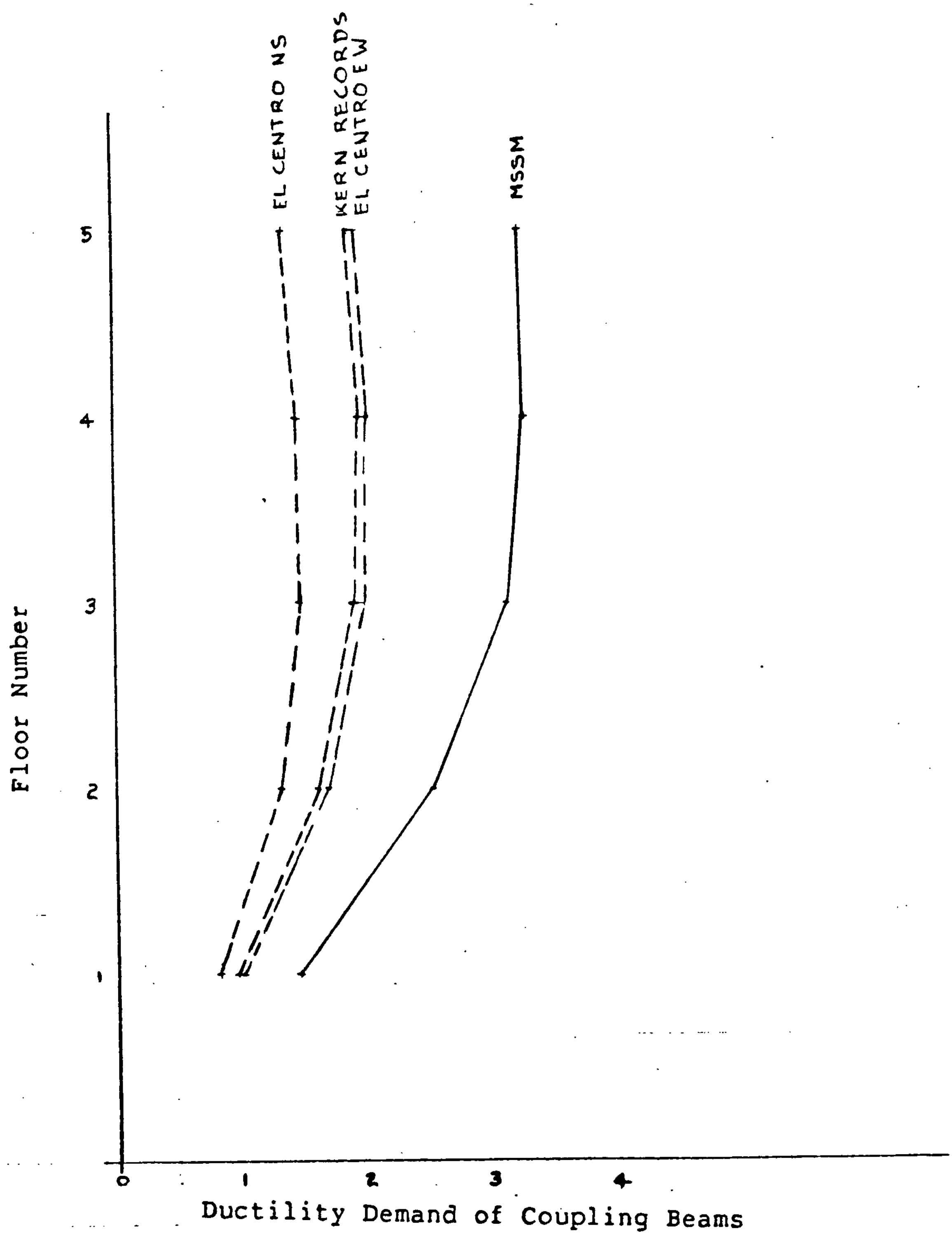


Figure 5.4
Displacerent Envelopes for the 5-Story Wall
(Test Series 'B')

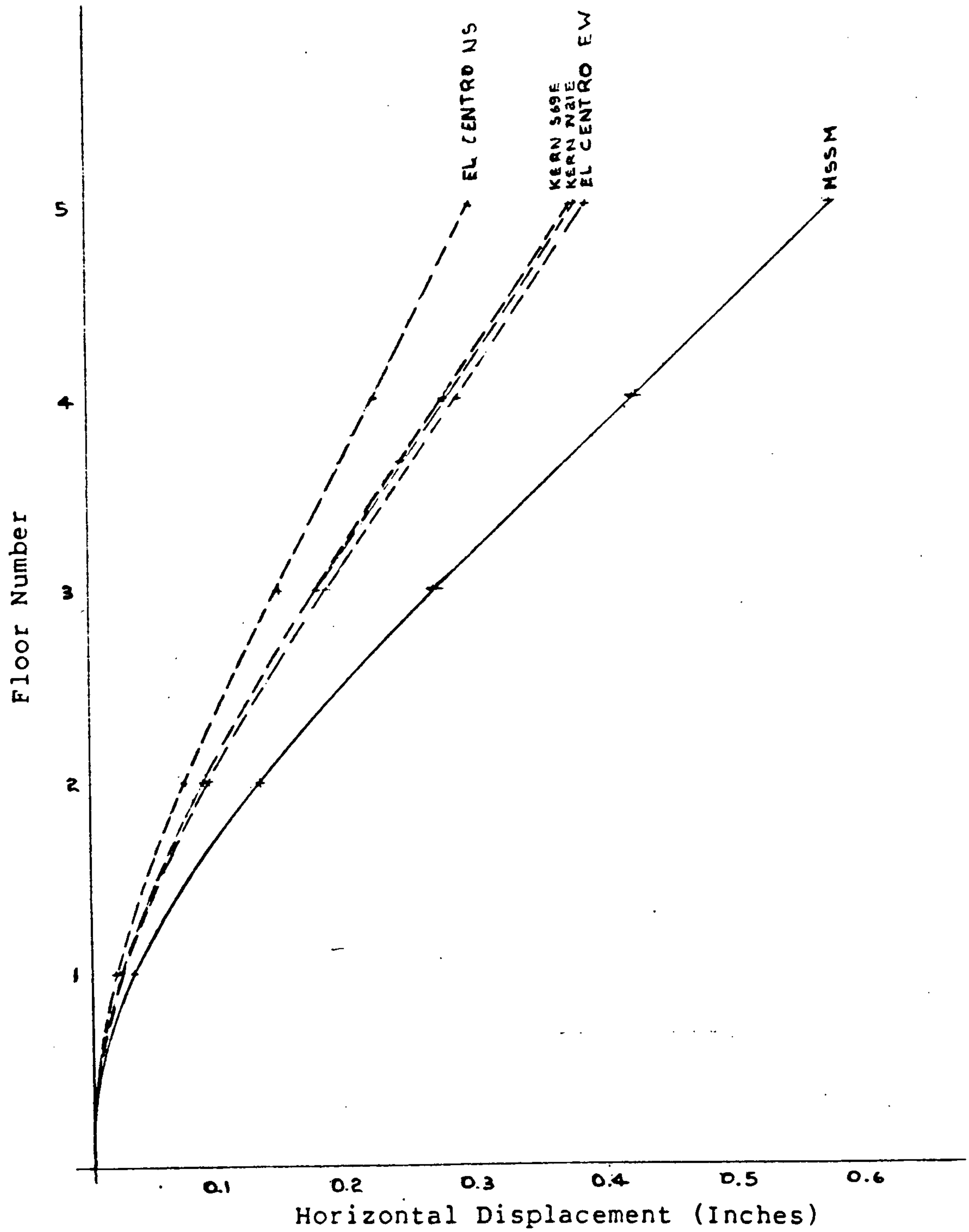


Figure 5.5
Ductility Demand Of the Coupling Beams for the 5-Story Wall (Mass=4 Times Original Run)

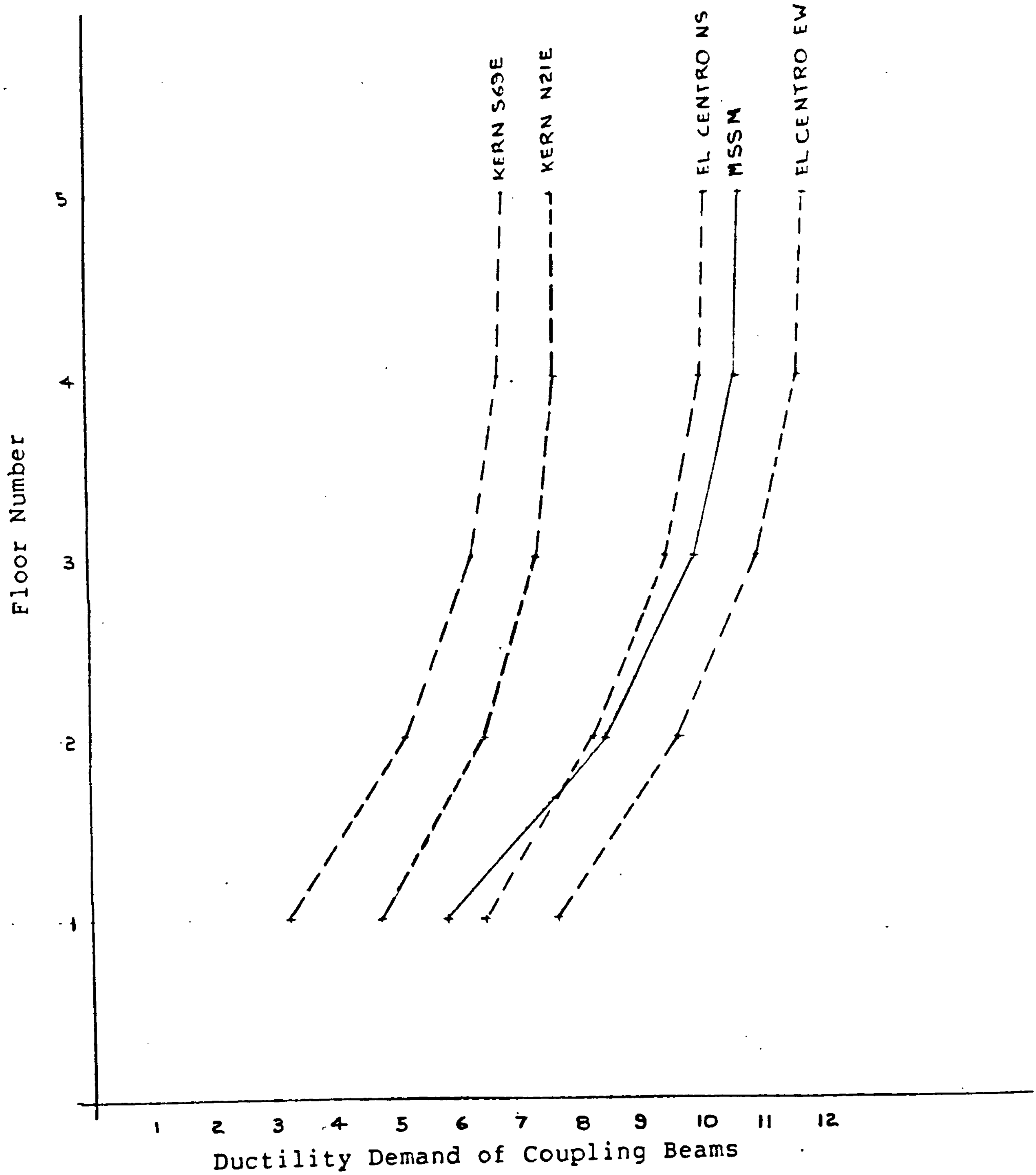
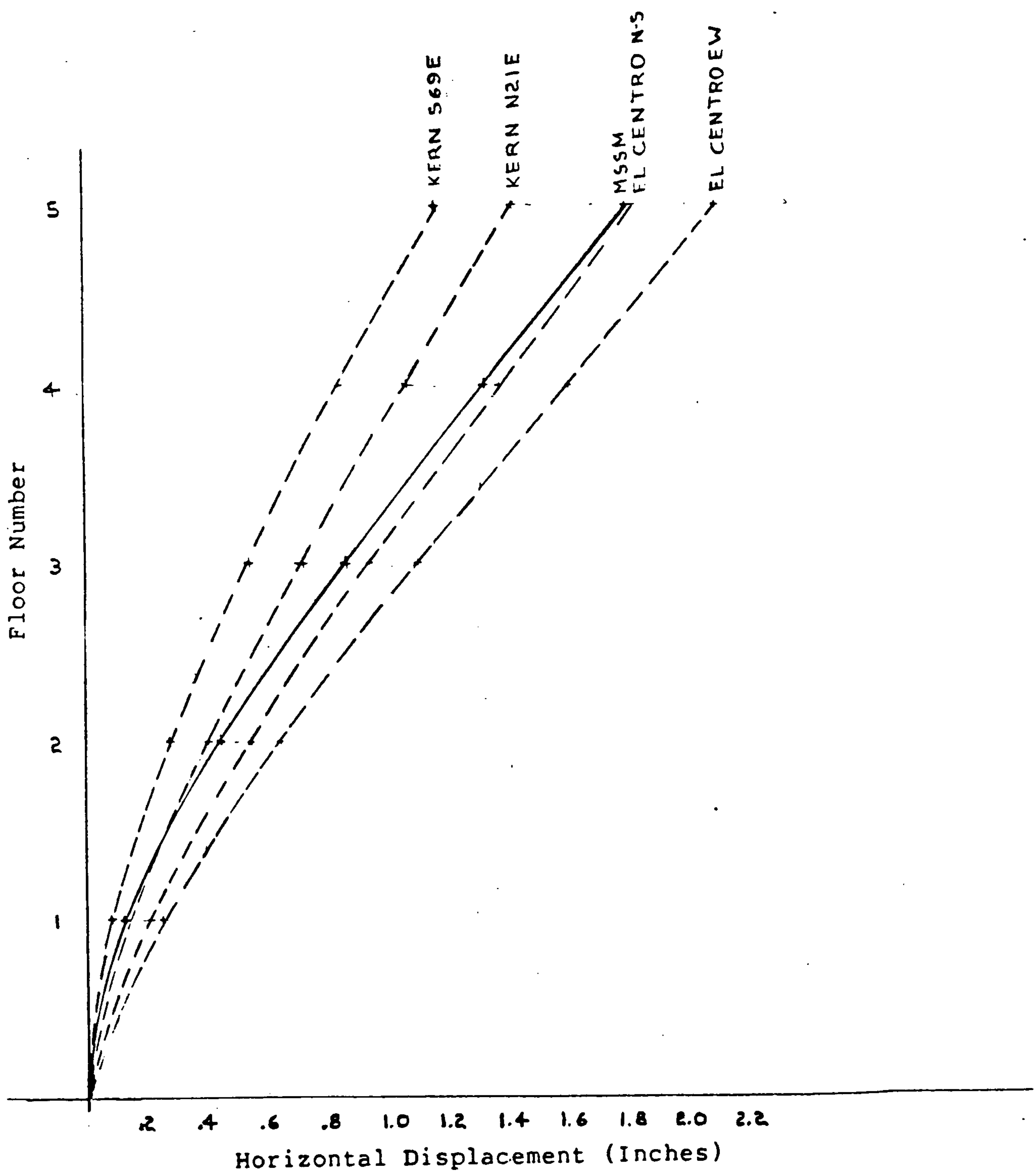


Figure 5.6
Displacement Envelopes for the 5-Story Wall
(Mass=4 Times Original Run)



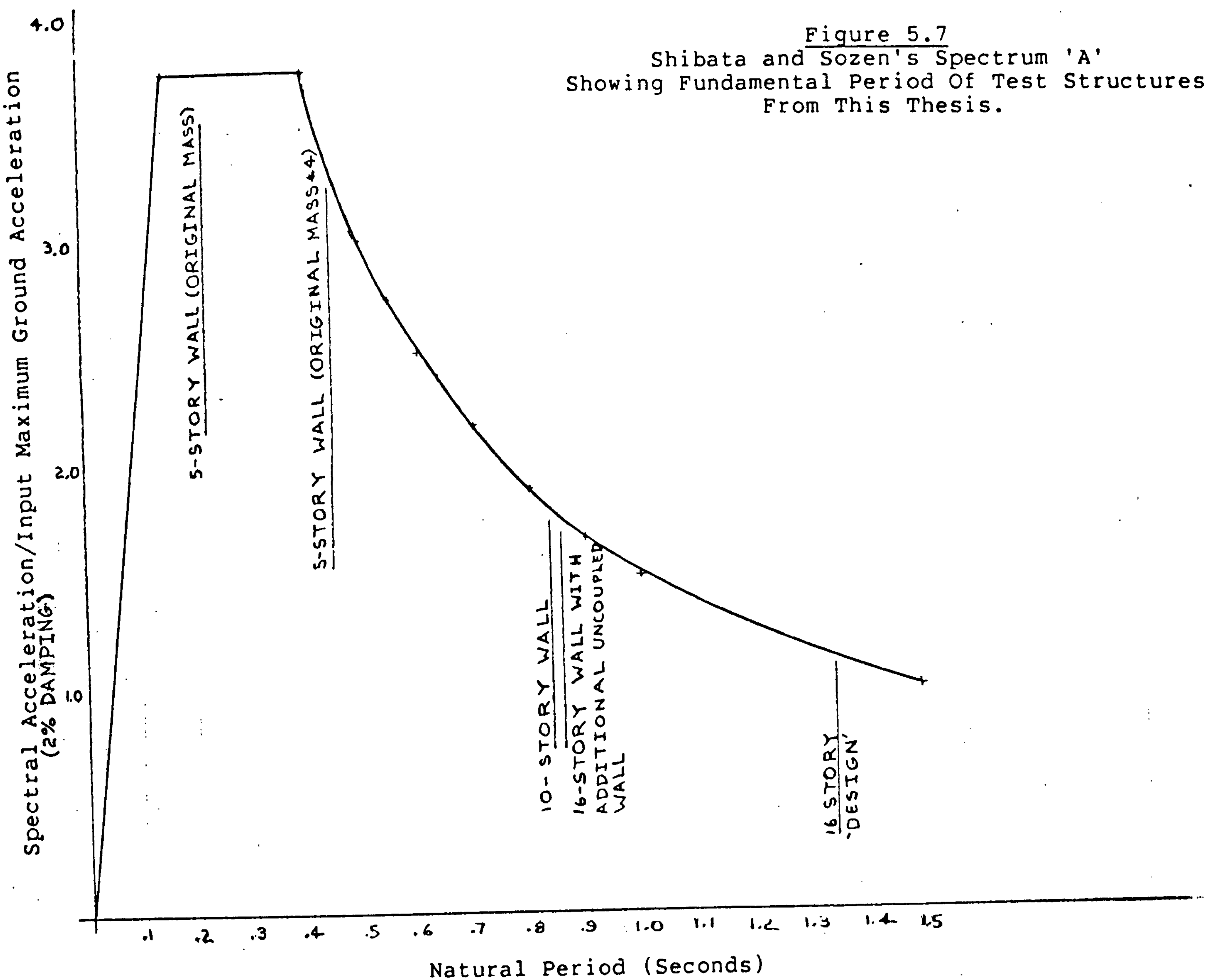
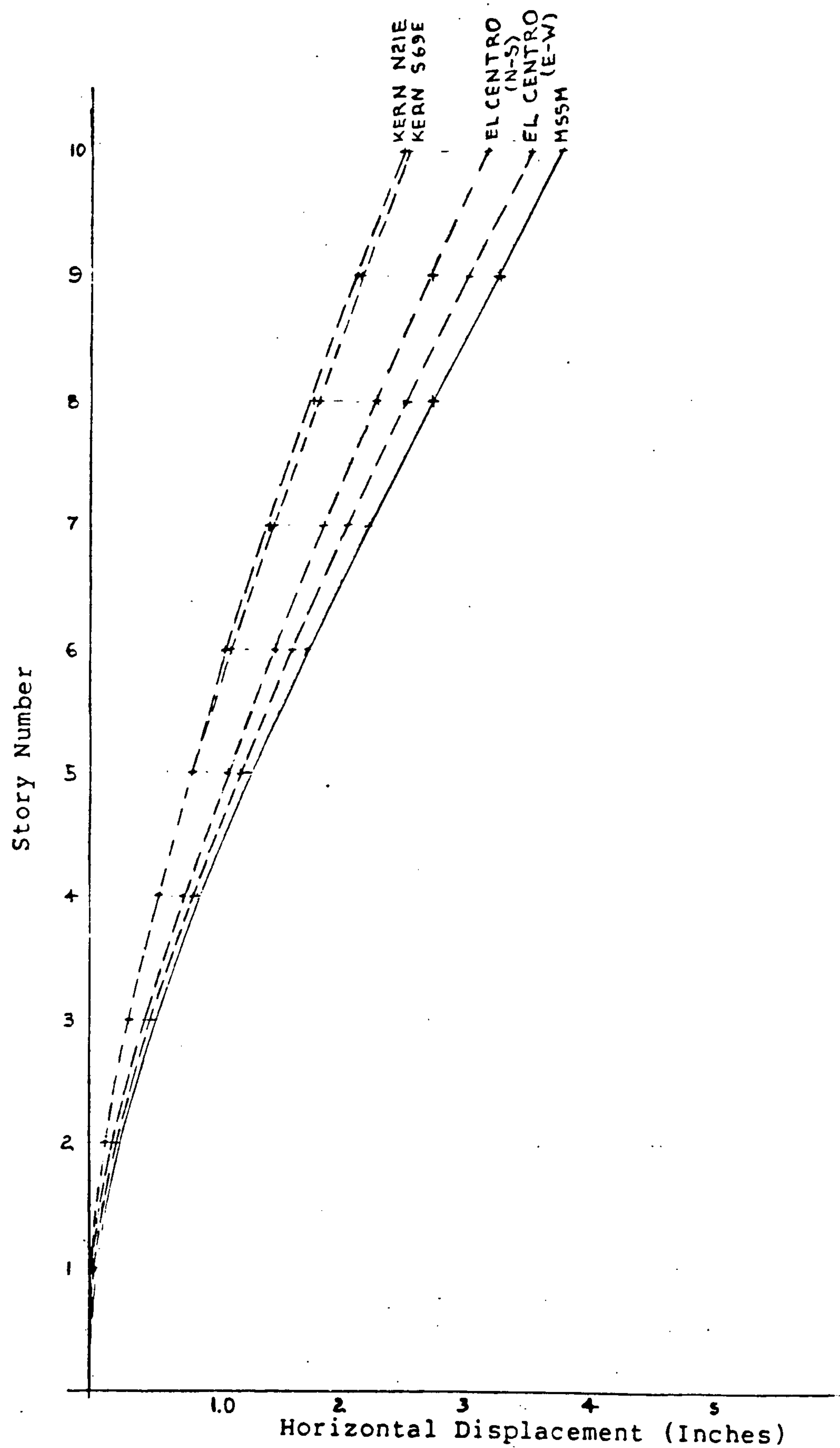
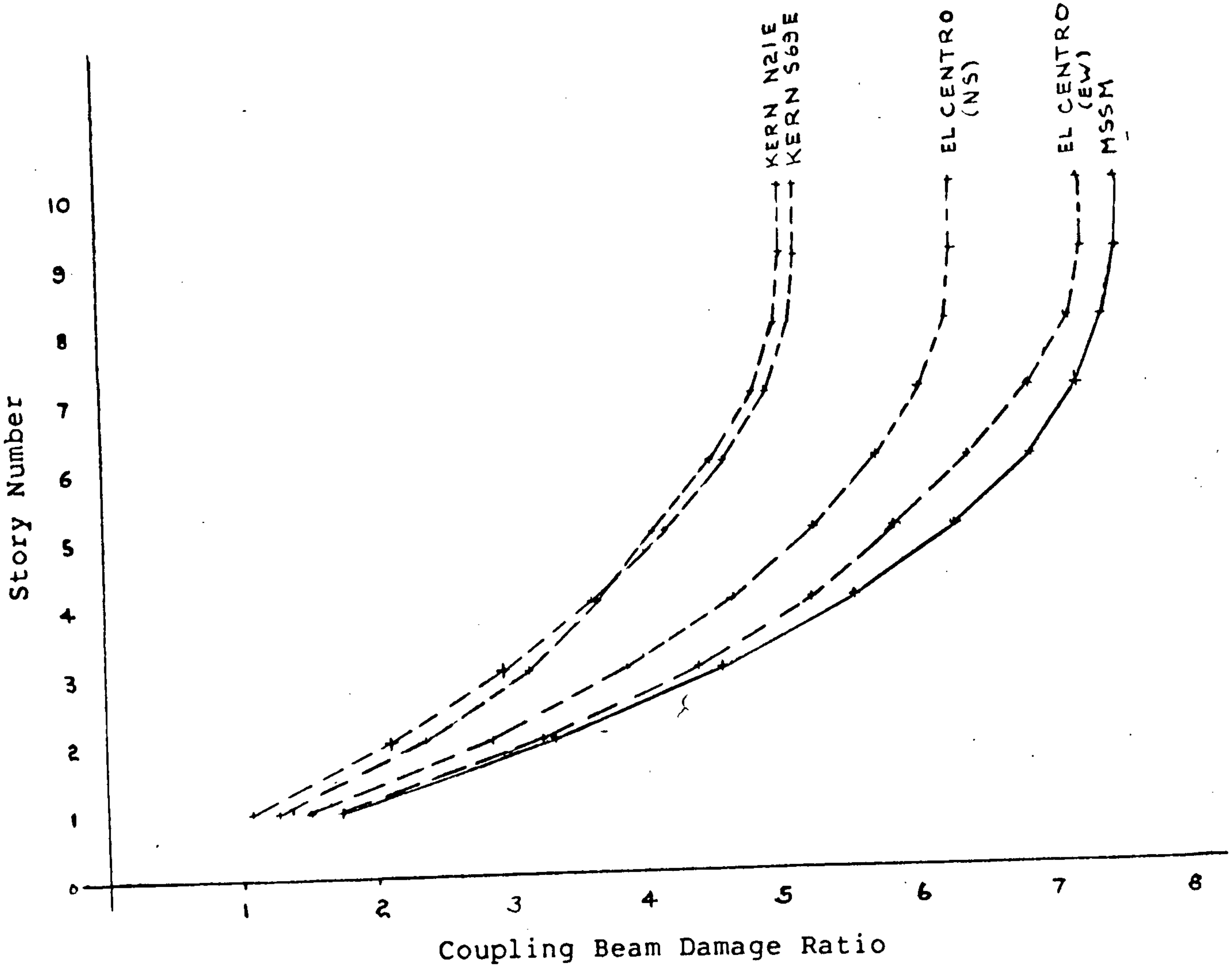


Figure 5.7
Shibata and Sozen's Spectrum 'A'
Showing Fundamental Period Of Test Structures
From This Thesis.

Figure 5.8
Deflection Envelopes For The 10-story Wall



<u>Figure 5.9</u>
Coupling Beam Damage Ratios For The
Ten Story Wall.



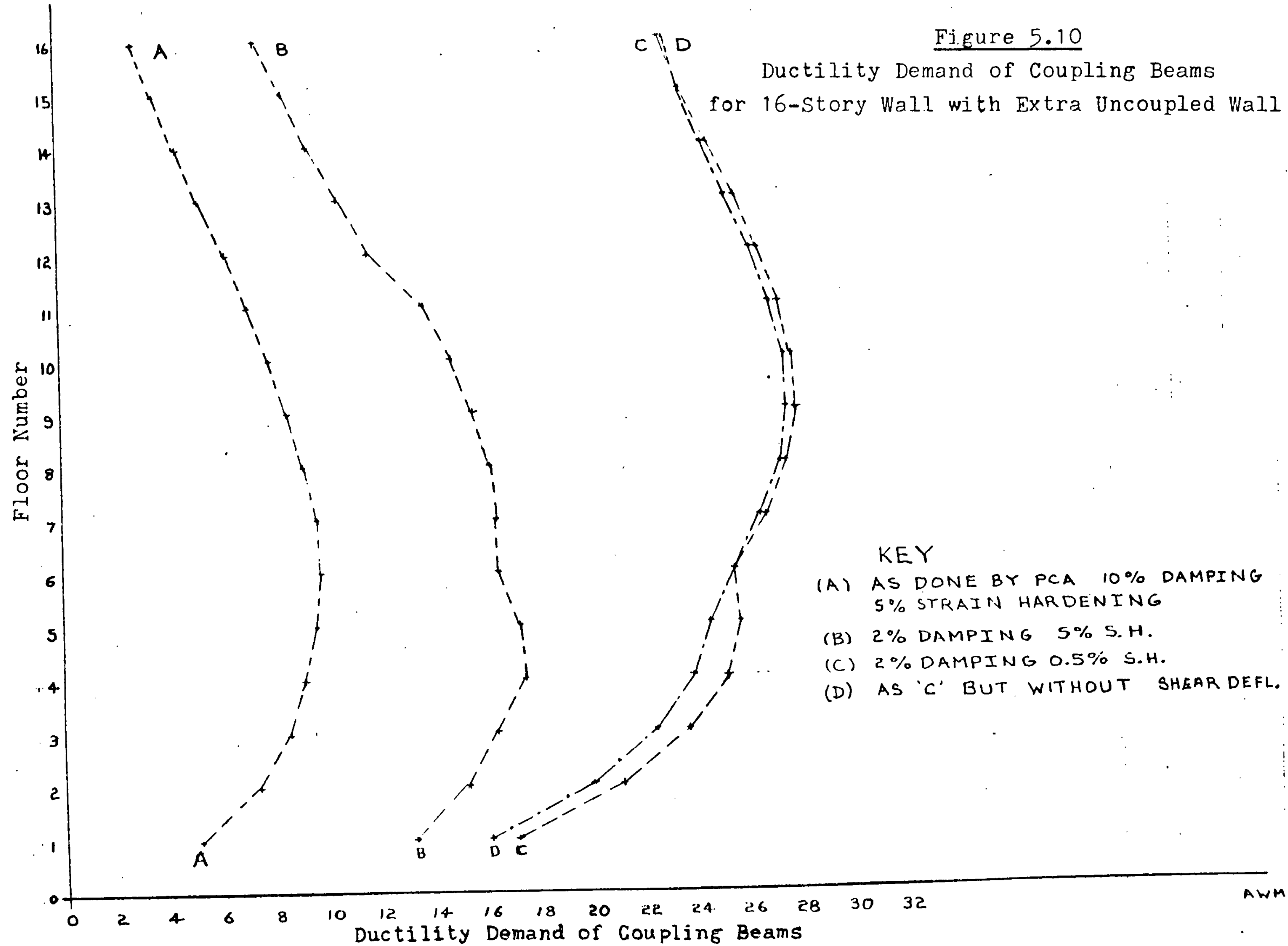


Figure 5.10
Ductility Demand of Coupling Beams
for 16-Story Wall with Extra Uncoupled Wall

Figure 5.11
Deflection Envelopes For The 16-story Coupled Wall With Attached Uncoupled Wall

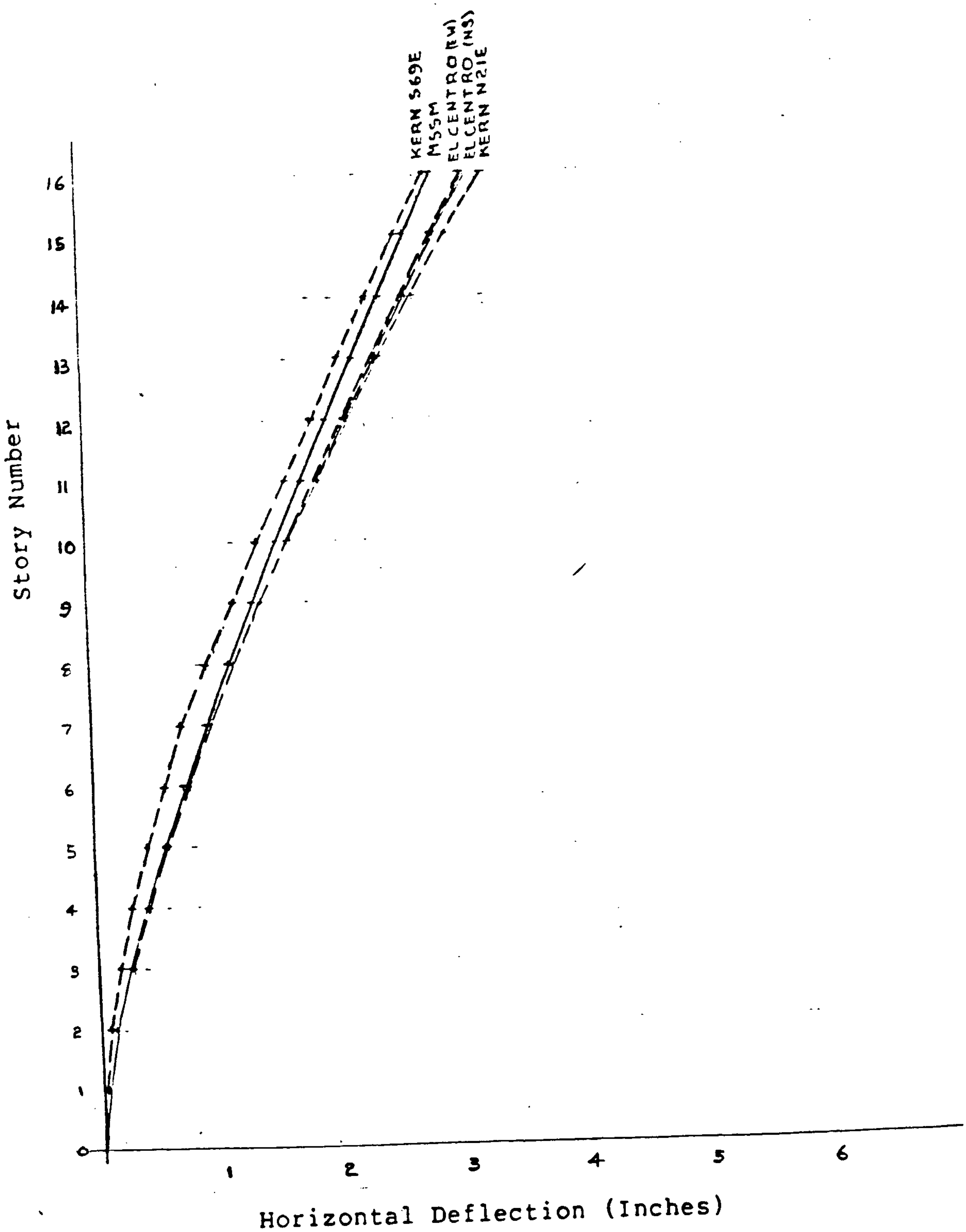


Figure 5.12
Damage Ratios For The 16-story
Coupled Wall With Attached Uncoupled Wall

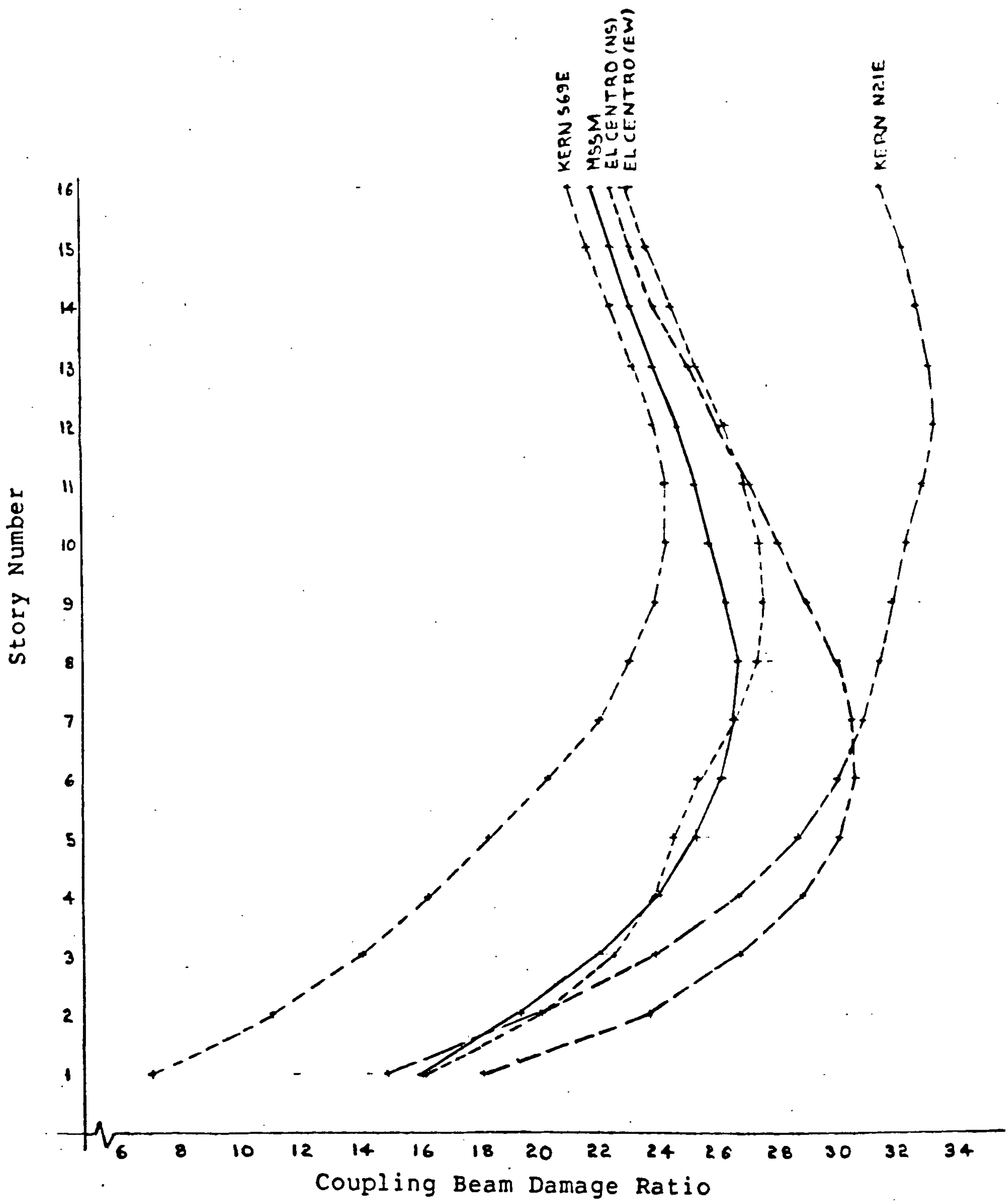


Figure 5.13
Average Ductility Demand of Coupling Beams for the 16-Story Coupled Wall with Attached Uncoupled Wall.

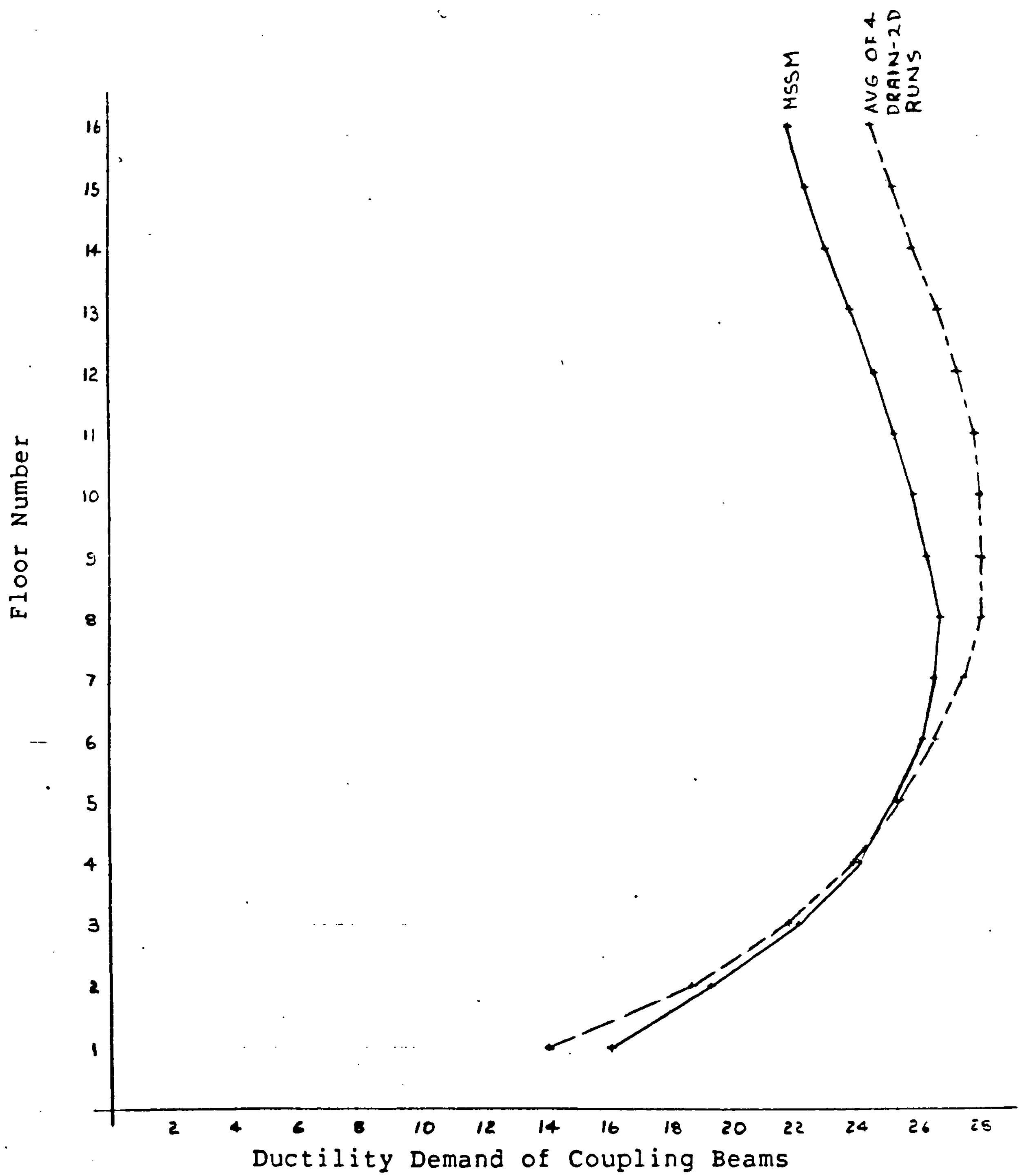
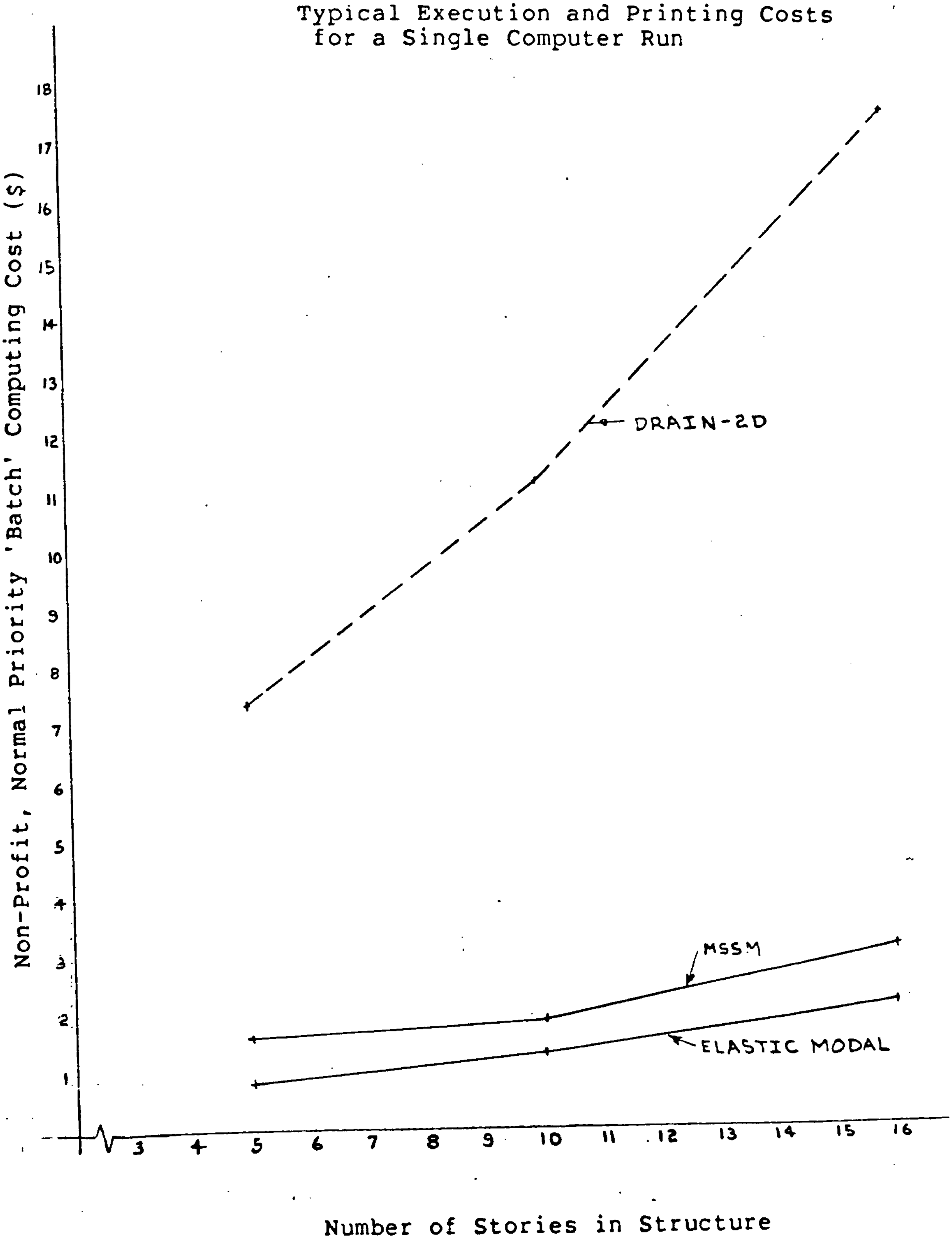


Figure 5.14
Typical Execution and Printing Costs for a Single Computer Run



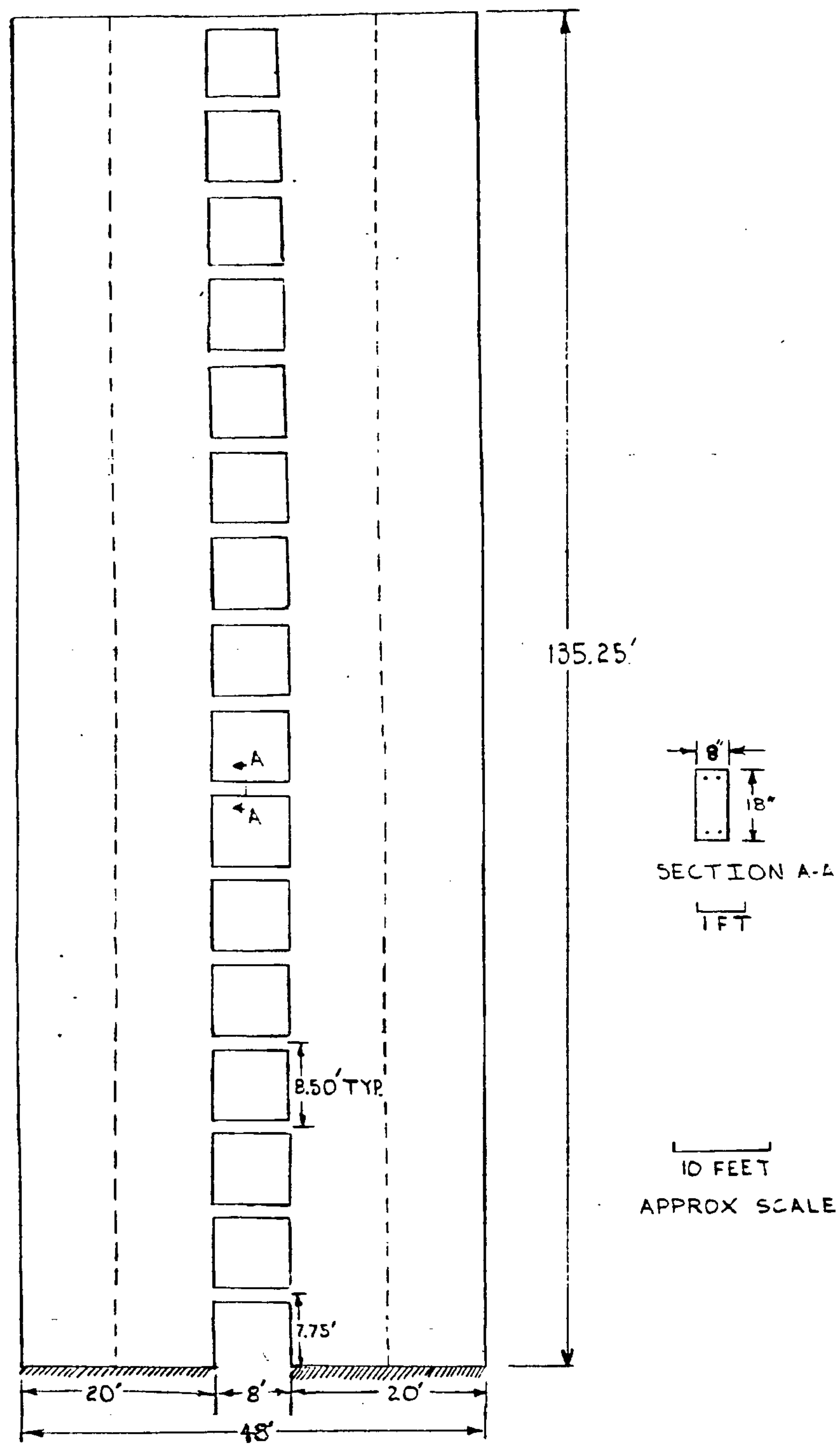


Figure 6.1
Configuration of 16-Story Coupled Wall for Design Example

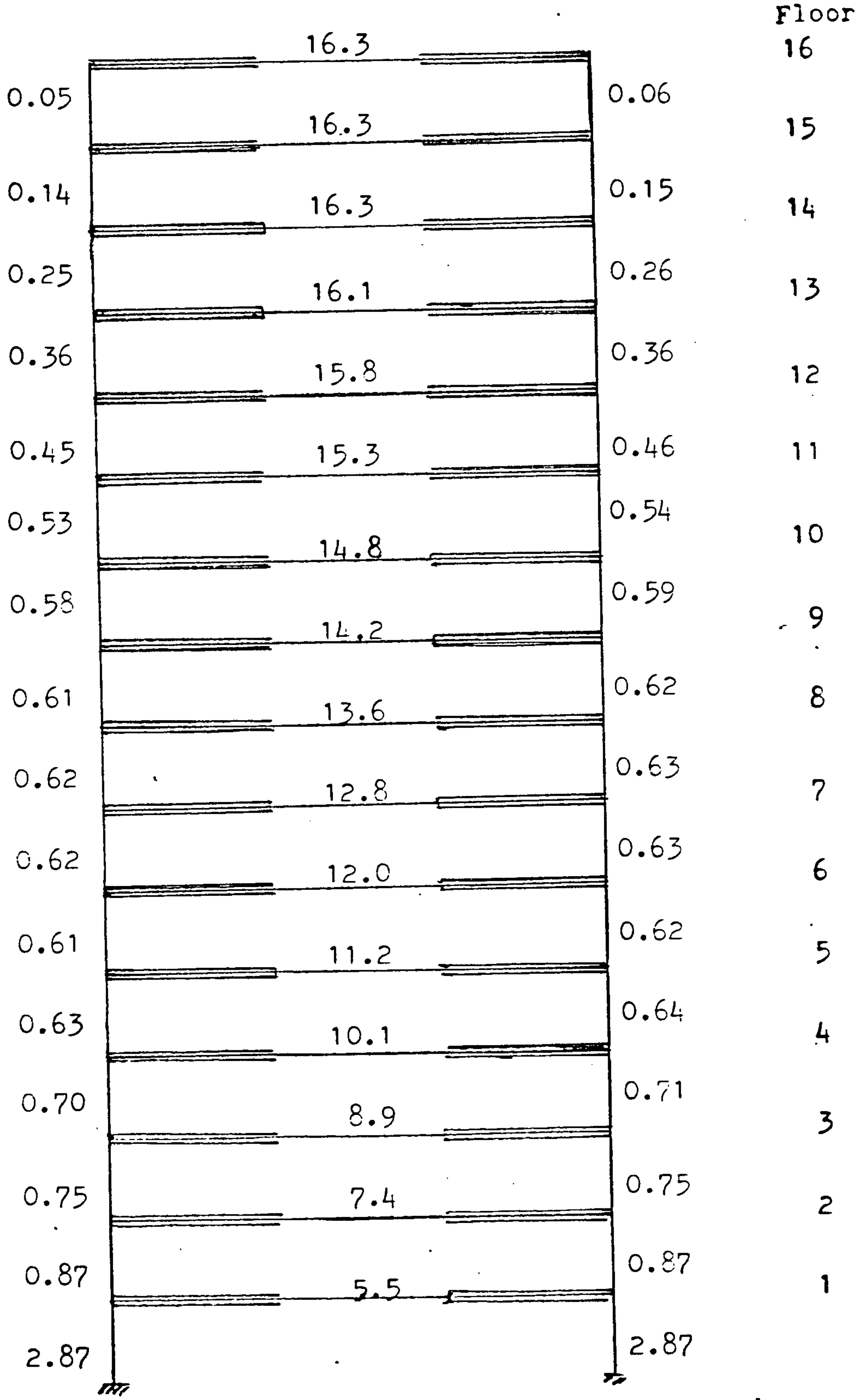


Figure 6.2
Damage Ratios from the First Run on the 16-Story Design Example

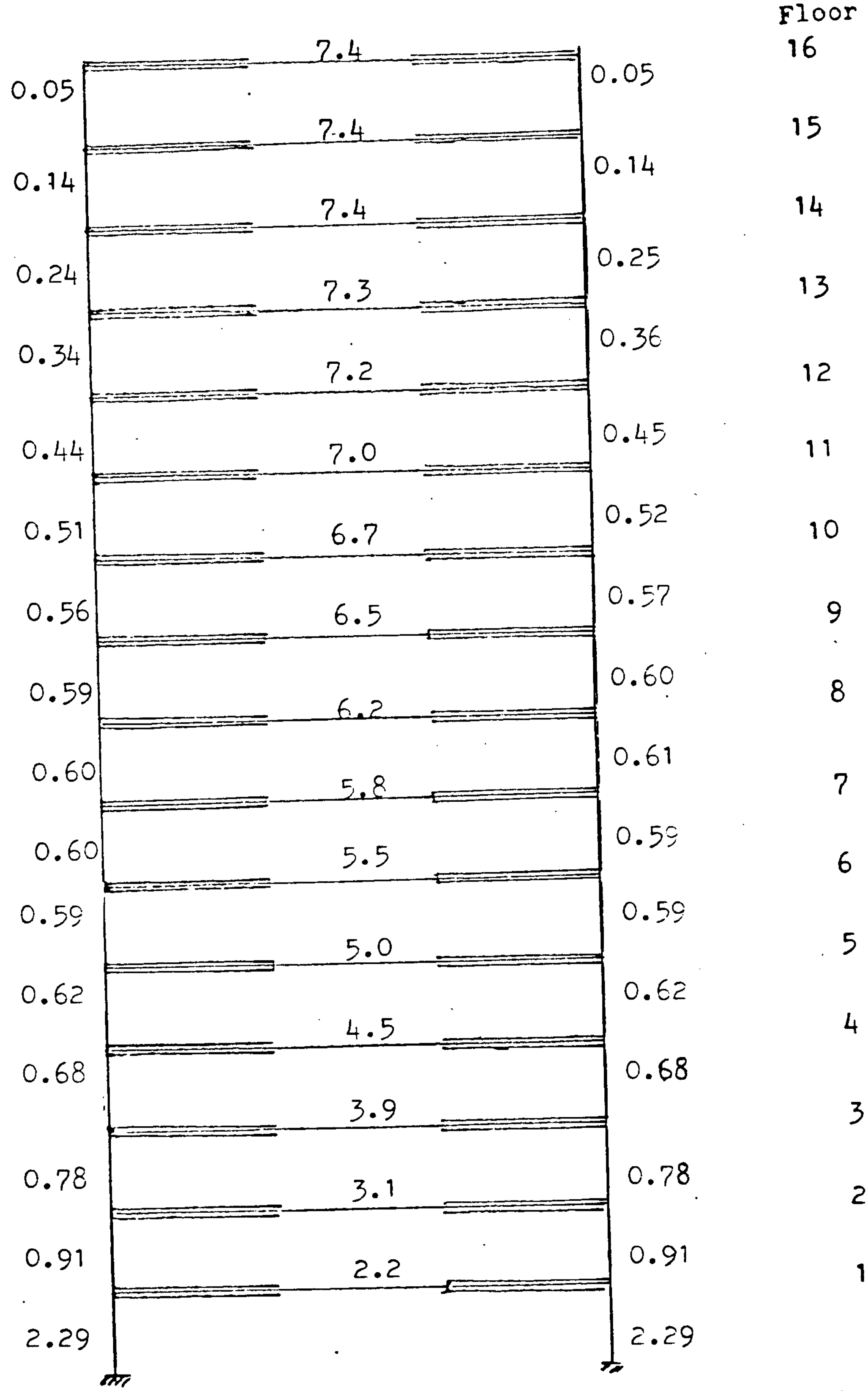


Figure 6.3
Damage Ratios from the Second Run on the 16-Story Design Example

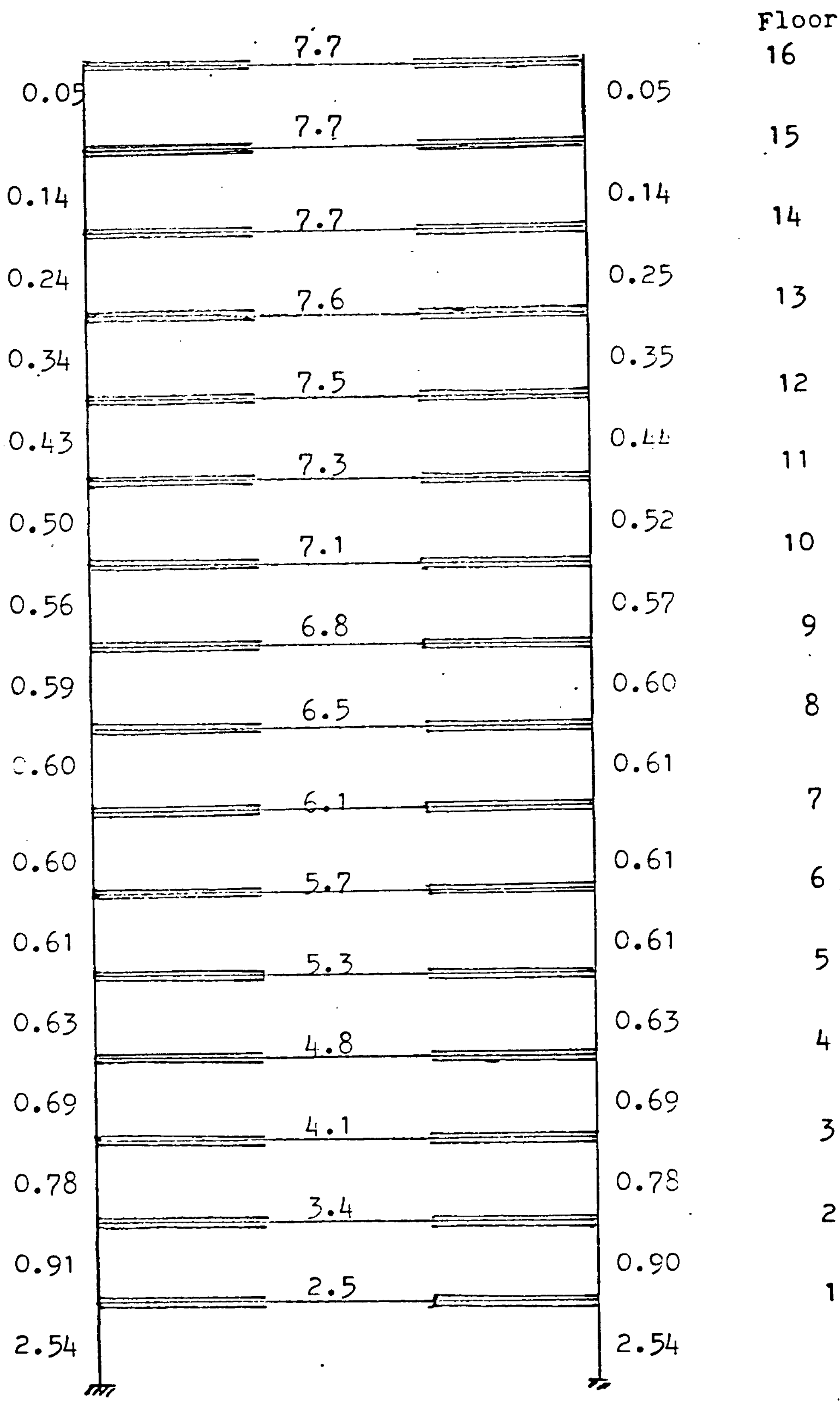


Figure 6.4
Damage Ratios from the Third Run on the 16-Story Design Example

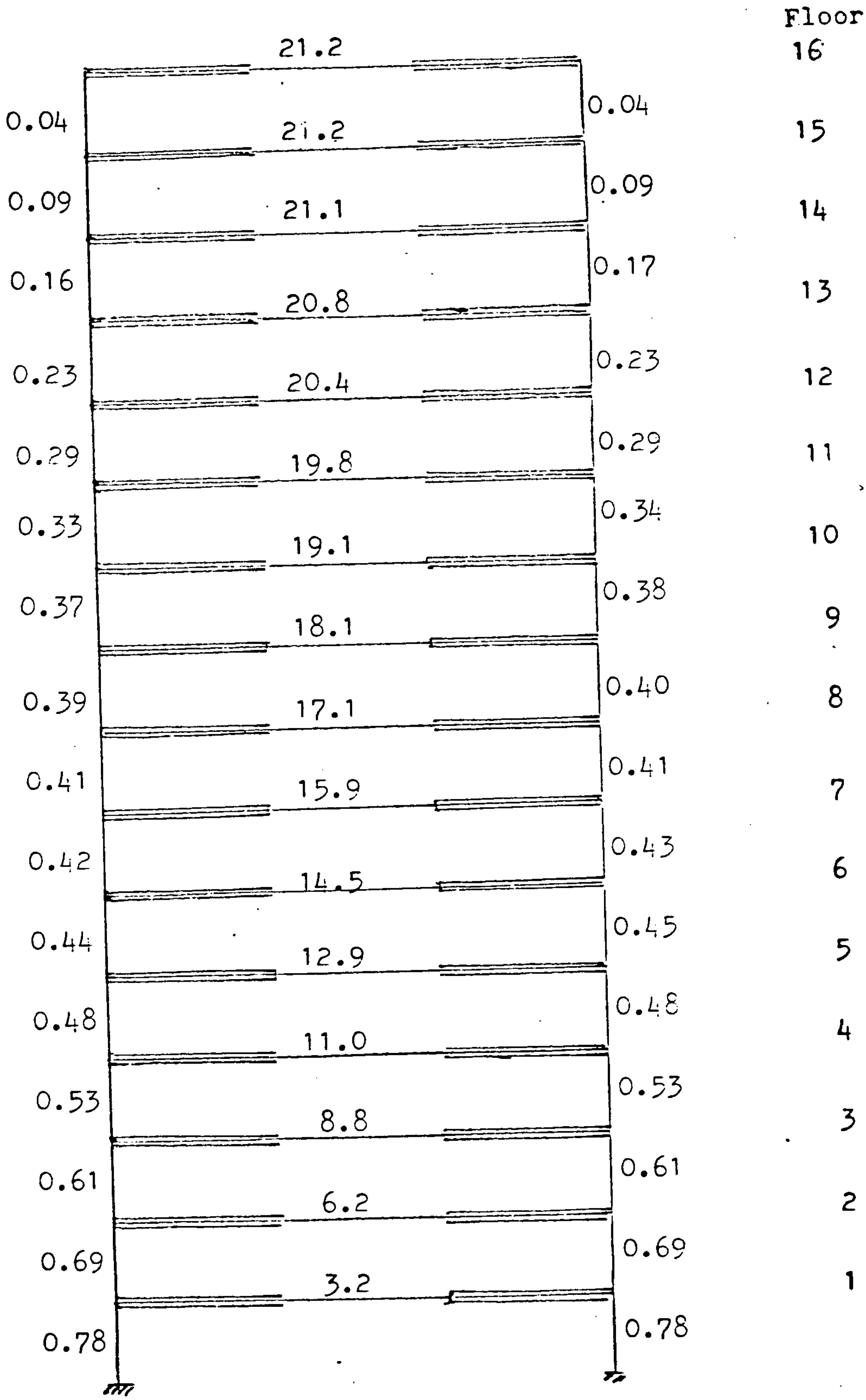


Figure 6.5
Damage Ratios from the Forth Run on the 16-Story Design Example

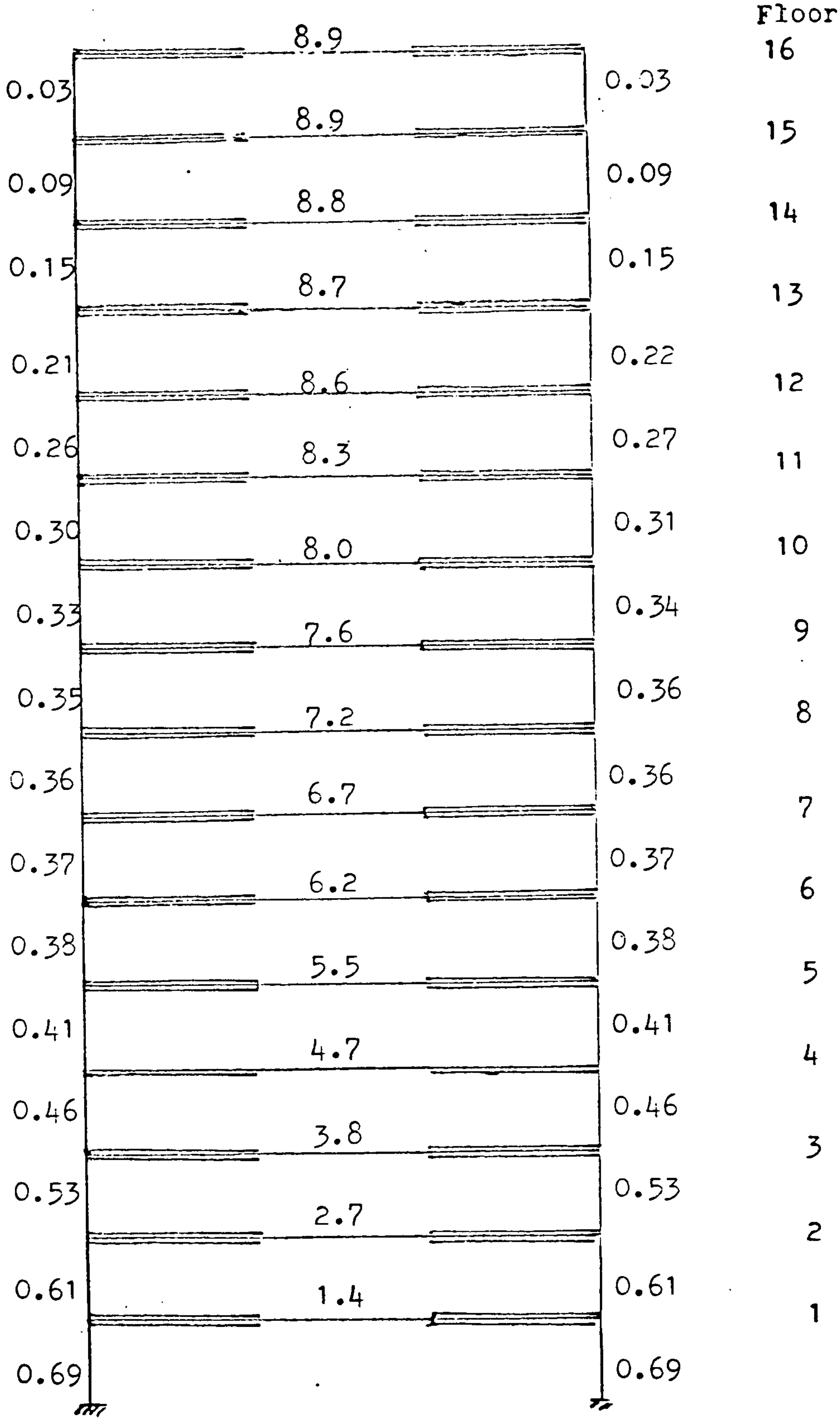


Figure 6.6
Damage Ratios from the Fifth Run on the 16-Story Design Example

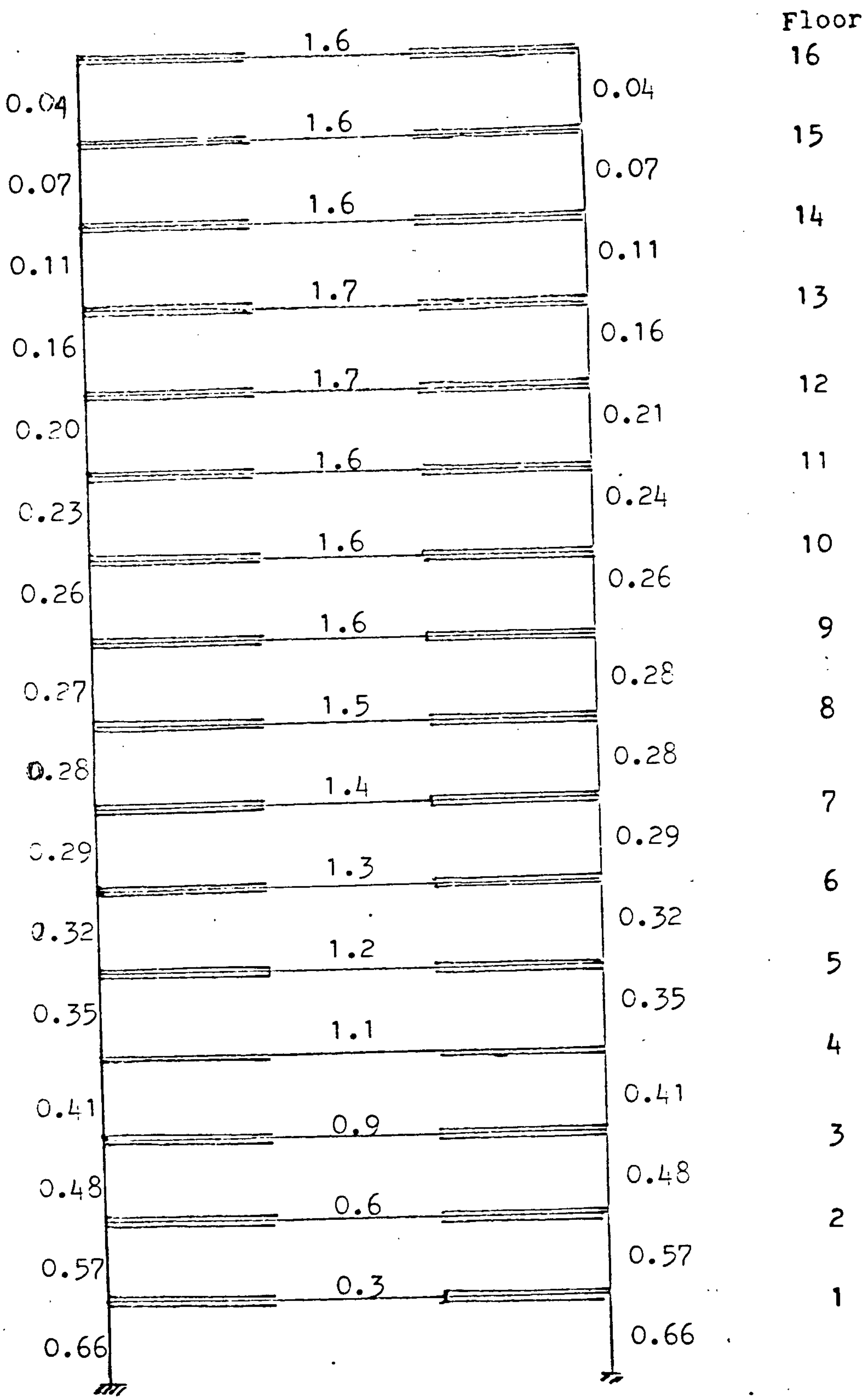


Figure 6.7
Damage Ratios from the Sixth Run on the 16-Story Design Example

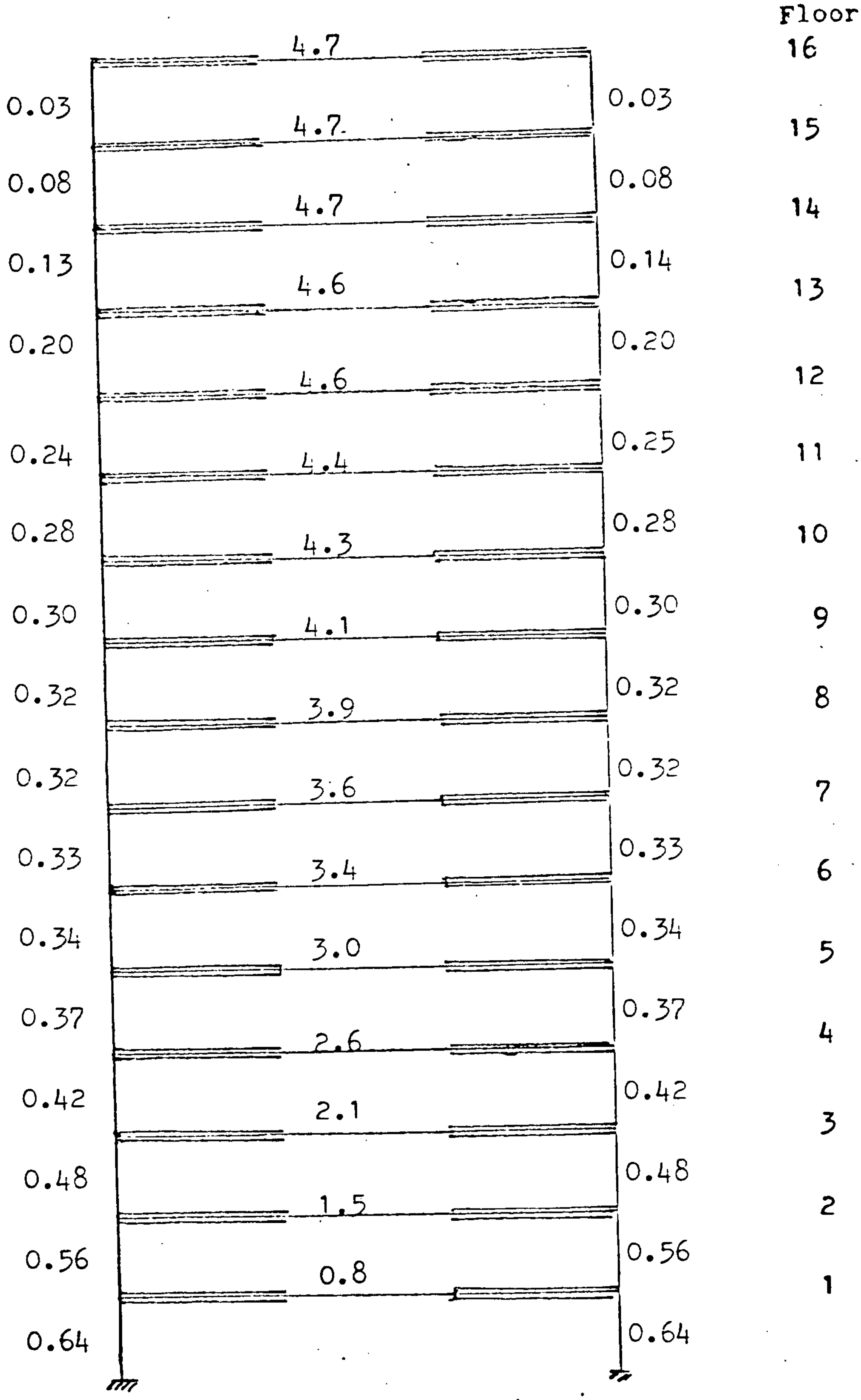


Figure 6.8
Damage Ratios from the Seventh Run on the 16-Story Design Example

Figure 6.9
Damage Ratios of Coupling Beams from DRAIN-2D Runs On 16-Story Design Example

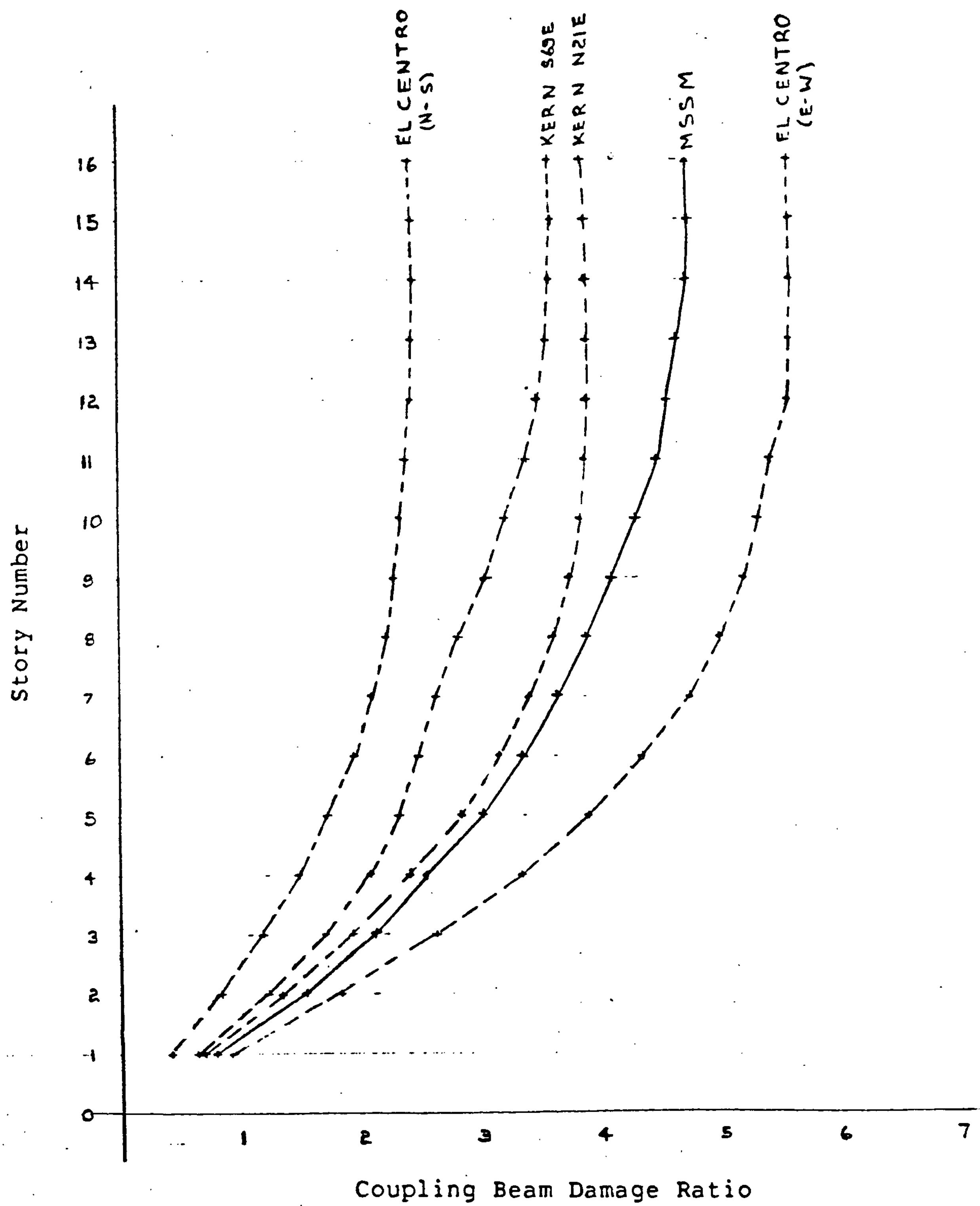
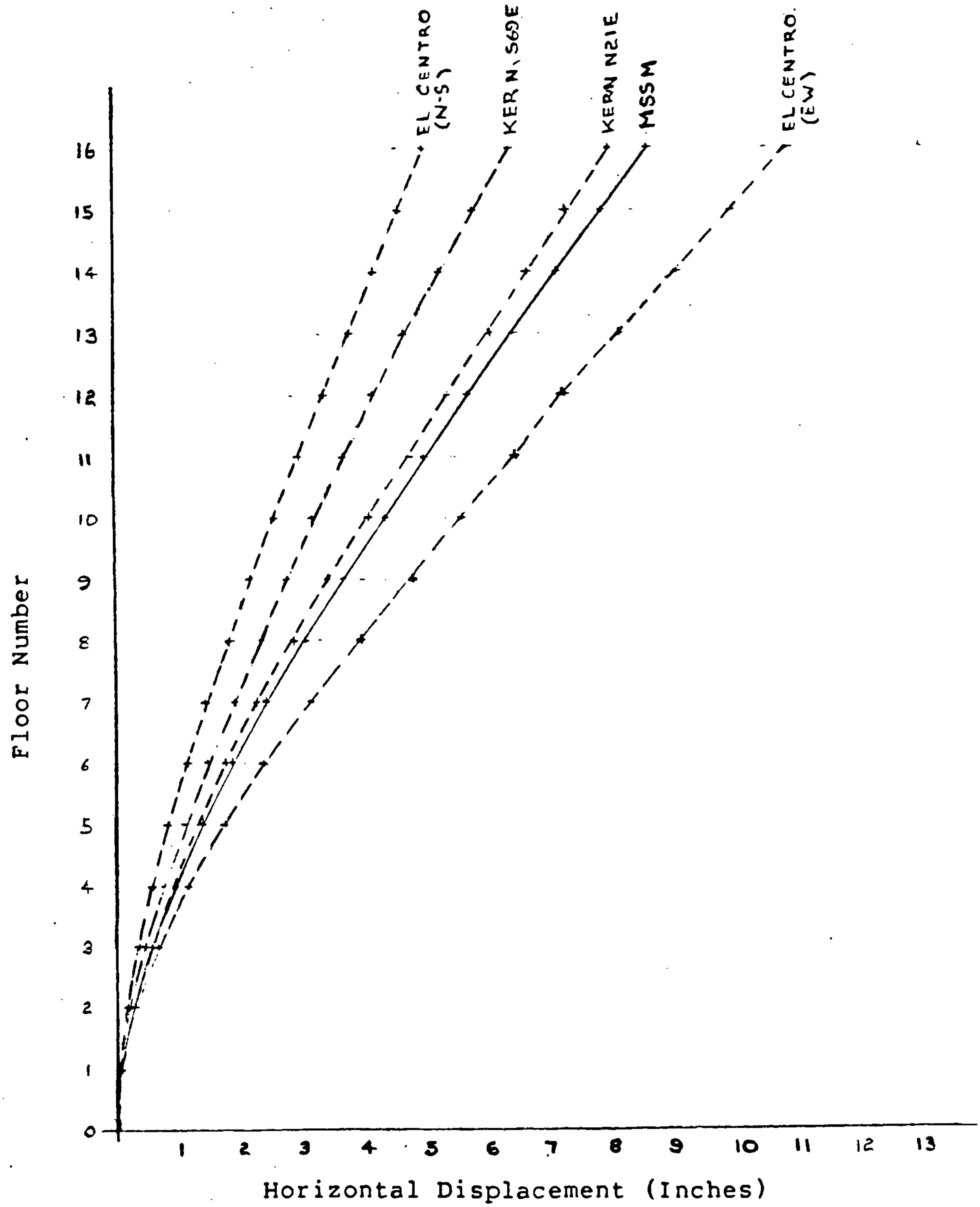


Figure 6.10
Deflection Envelopes from DRAIN-2D
Runs On 16-Story Design Example



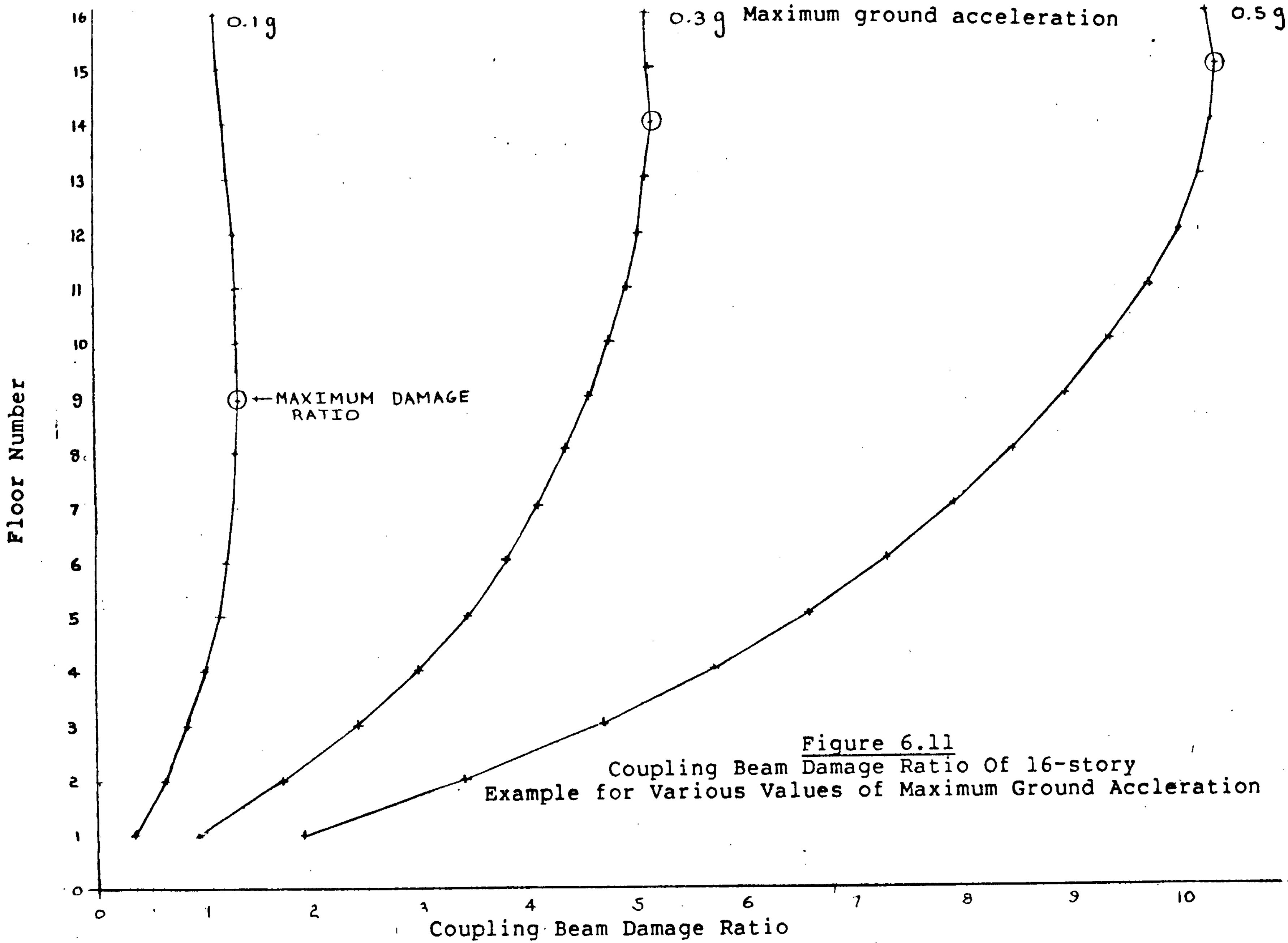


Figure 6.11
Coupling Beam Damage Ratio Of 16-story
Example for Various Values of Maximum Ground Accleration

Figure 6.12
Damage Period as a Function of Maximum Ground Acceleration for the 16-Story Example.

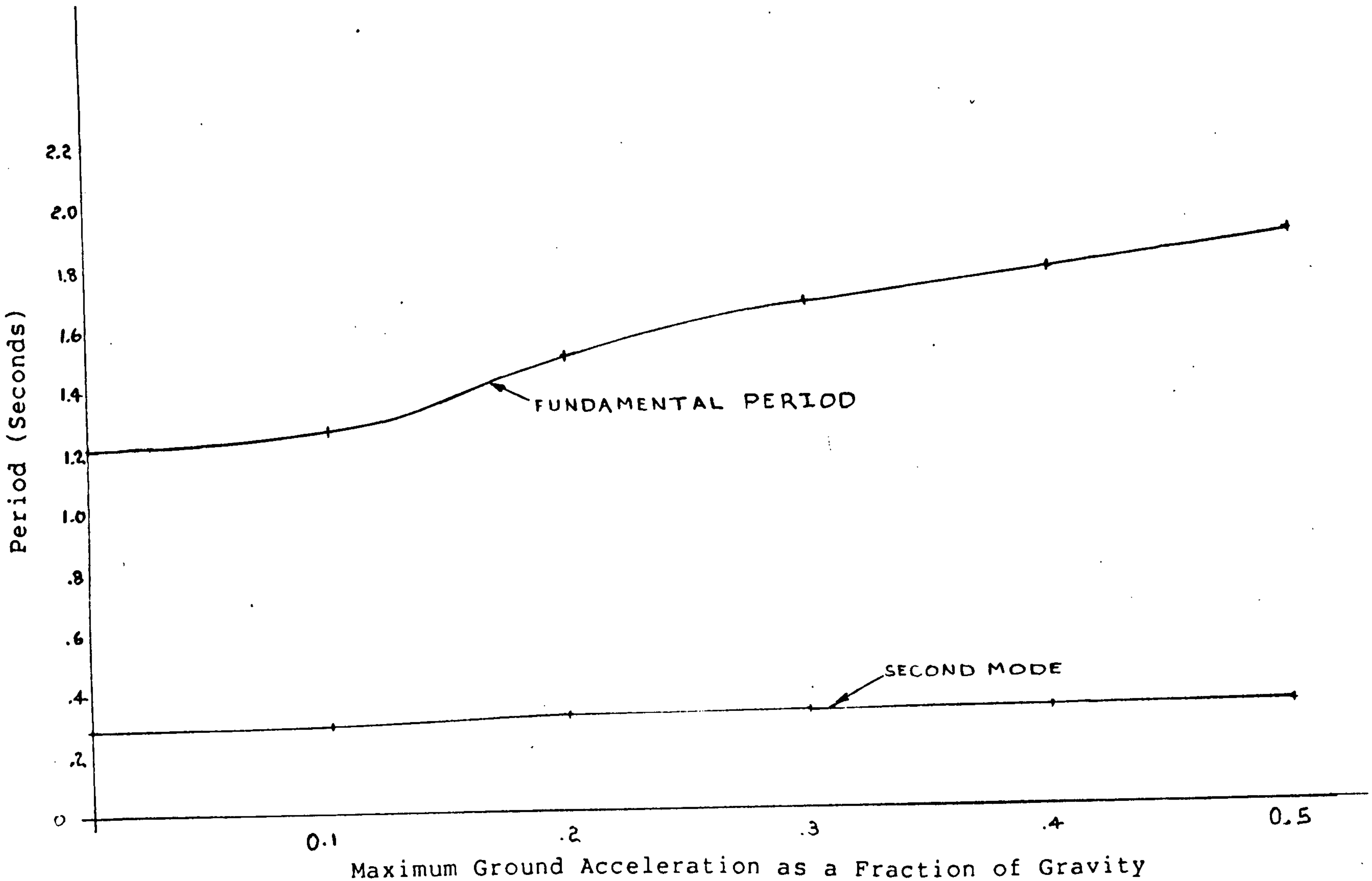
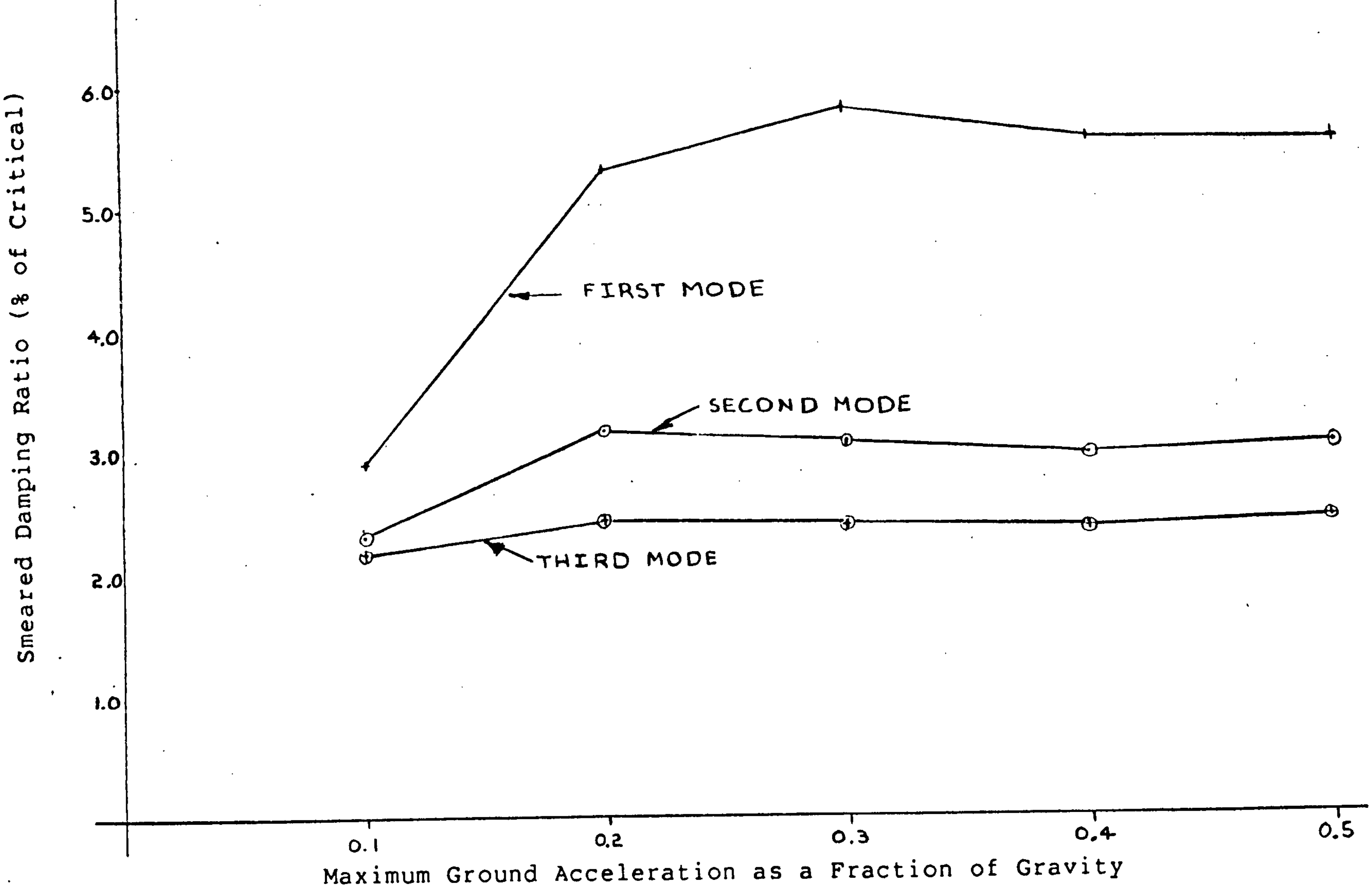
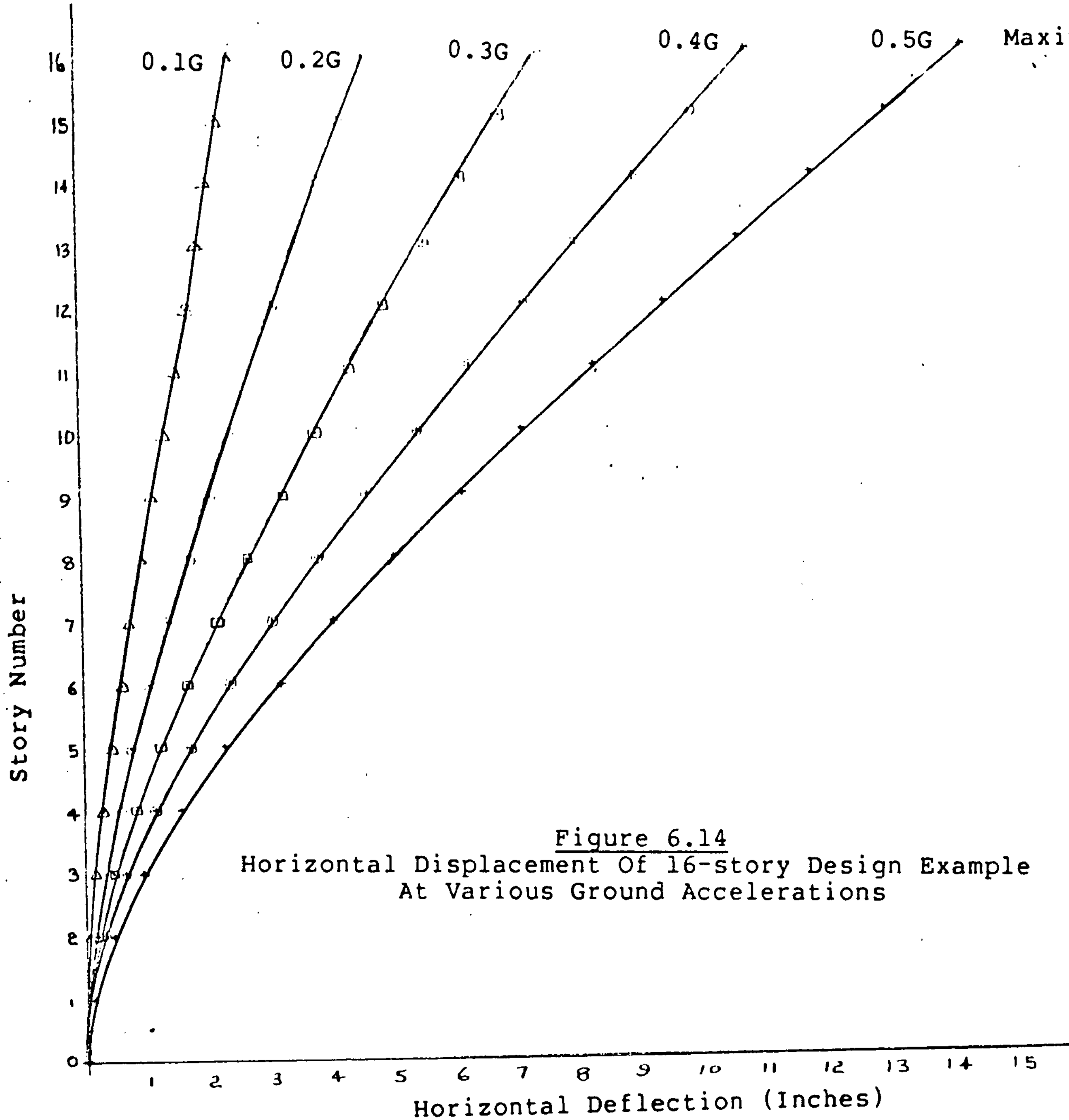


Figure 6.13
Speared Damping Ratio as a Function of Maximum Ground Acceleration for the 16-Story Example.





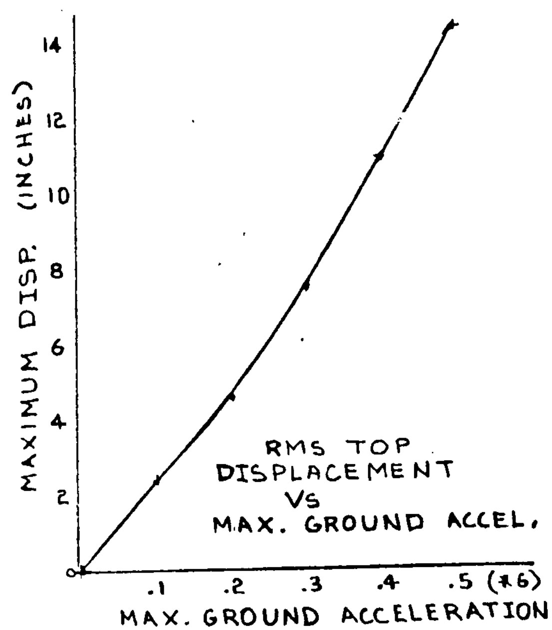


Figure 6.14
Horizontal Displacement Of 16-story Design Example
At Various Ground Accelerations

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(2) Fintel, Mark and Ghosh, S.K. Seismic Resistance of a 16-Story Coupled-Wall Structure: A Case Study Using Inelastic Dynamic Analysis. Portland Cement Association, 1980.

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(4) Paulay, Thomas "An Elasto Plastic Analysis of Coupled Shear Walls." ACI Journal, November 1970, pp 915-922.

(5) Paulay, Thomas "Design Aspects of Shear Walls for Seismic Areas". Canadian Journal of Civil Engineering 2,321 (1975).

(6) Paulay, T and Uzumeri, S.M. "A Critical Review of the Seismic Design Provisions for Ductile Shear Walls of the Canadian Code and Commentary". Canadian Journal of Civil Engineering. 2, 592, (1975) pp 592-601.

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(8) Shibata, Akenori and Sozen, Mete. A. "Substitute-Structure Method for Seismic Design in R/C. Journal of the Structural Division ASCE Jan 1976, pp 1-18.

(9) Structural Analytical Section, Engineering Development Department Portland Cement Association instructions for preparing input data for DRAIN-2D. May 1978.

(10) Yoshida, Sumio Modified Substitute Structure Method for Analysis of Existing R/C Structures. Master's thesis University of British Columbia 1979.

USER'S MANUAL

ELASTIC AND/OR DAMAGE AFFECTED MODAL ANALYSIS

(Utilizing the Modified Substitute Structure Method.)

Program Name: EDAM

DISCLAIMER:

EDAM

PROGRAM HISTORY

UPDATES	MODIFICATIONS	PROGRAMMER
1978	MSSM.S program written	Sumio Yoshida
Aug 1979	MSSM.S manual written	Ron Grig
1980-1981	MSSM.S to EDAM	Andrew W. F. Metten
Sept-Oct 1980	EDAM manual written	Andrew W. F. Metten.

NOTE the program was renamed from its original title of MSSM.S to distinguish it from the orignal program in the Civil Engineering Program Library and to reflect the different capabilities and *modus operandi* of the new program. EDAM is 1646 lines long while MSSM.S is 824 lines long.

TABLE OF CONTENTS

INTRODUCTION

EDAM stands for Elastic and/or Damage Affected Modal analysis.

This program performs elastic and elastoplastic analysis of plane frames. The elastoplastic analysis is performed using the Modified Substitute Structure Method. The program is cabable of handling all combinations of fixed and pin ended beams as well as fixed ended beams which have rigid extensions.

EDAM was originally written by Sumio Yoshida and called MSSM.S upon completion. The program was renamed to EDAM after being extensively modified during a subsequent master's thesis research project during 1980. Eventually both EDAM and MSSM.S will be found in the Civl Engineering Program Library.

In developing the computer program which could apply the modified substitute structure method to a structure, considerable effort was applied to make it a method that could be used by the practising engineer in a design situation. At the expense of slightly increasing the storage requirements and execution time 'common blocks' were not utilized in writing of the code. This was done in the belief that a computer program is only complete when it it is found to be too unwieldy to modify to tackle a problem varying in some form from the originally designed task. A program without large common blocks will often be easier to modify as each subroutine has more autonomy from the remainder of the program. By eliminating common blocks the modification of the program is made easier. Should anyone try to modify the program EDAM they will find a large collection of 'comment cards' in the program outlining what is being attempted at each stage as well as detailing what many of the variable names stand for. When modifying the program the concept throughout was to eliminate all unnecessary complexity. While it is possible to build structure data generators into a program this often creates unnecessary complexity. Data generation programs are a valuable asset to the speedy computer analysis of a structure; however it was the belief in modifying the program that they are better kept seperate from the program that is to use the data. Besides making the main program simpler, this system has several other advantages. If the generator is separate from the execution of that data, editing of the data can occur before its execution which extends the capabilities of the generator. Another advantage is that recompilation costs are reduced if the source code is altered when there are fewer executable lines.

THEORY

The program uses the spectrum tehnique for calculating forces and displacements. Modal analysis is carried out to produce the results that are printed. The user should be familar with modal analysis to fully appreciate the output. Further details may be found in Sumio Yoshida's master's thesis "MODIFIED SUBSTITUTE STRUCTURE METHOD FOR ANALYSIS OF EXISTING R/C STRUCTURES" March 1979 and Andrew W.F.Metten's thesis: The Modified Substitute Structure Method As A Design Aid For Seismic Resistant Coupled Structural Walls, March 1981.

PROGRAM RESTRICTIONS

Most of the program restictions that will affect the user can be found in Sumio Yoshida's thesis. These points are reproduced here as a reminder.

The following restrictions apply to both elastic and damage affected runs.

(1) The system can be analyzed in one vertical plane.

(2) There are to be no abrupt changes in mass, stiffness or geometry throughout the height of the structure.

(3) Nonstructural components are to be such that they do not affect the response of the system as modeled.

(4) 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)

(5) 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.

(6) The structure must comply with the dimensioning requirements of EDAM.

The following restrictions apply <u>only</u> to damage affected analysis.

(1) The materials used for the construction of the structure must be concrete. The development of the stiffness degredation and damping formula was done completly 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 initial fundamental elastic period should be such that it places the structure on a segment of the spectrum used which causes a decrease in the spectral acceleration when the period of the structure increases.

DIMENSIONING LIMITS

The maximum dimensions of the structure are.

100 members
100 joints
50 assigned masses
10 eigenvalues
(total degrees of freedom)*(half bandwidth) is less than 2000.

These dimensions can be easily increased by internal adjustment of the program should this be desired.

INPUT

Input to the program consists of program options and structure data. The units are British. The joint coordinates to be input in feet and decimals of a foot. Weights are in kips. Material constants in kips per square inch. The inertia of the sections to be inserted in inches to the fourth and the cross sectional areas in square inches. All input data is echo printed by the program.

CARD 1: INELAS,NMODES,NPRINT,ISPEC,AMAX,DAMPIN

Format(4I5,2F10.5) (one card)

INELAS =0 if elastic analysis is requested.
=1 or greater, if inelastic analysis is requested. INELAS should be set to the maximum number of inelastic iterations that may be performed before the program halts. A value of 50 should be sufficient.

NMODES = the number of modes to be included in the analysis and should be less than or equal to 10.

NPRINT = the number of modes for which printed displacements and forces are requested. Mode 1 to mode NPRINT inclusive will be printed. If NPRINT is greater than NMODES, then NPRINT will be set equal to NMODES. If NPRINT equals zero then only root-mean-square forces and displacements will be printed.

ISPEC =the spectrum type that is required.

=1 spectrum 'A' from the work of Shibata and Sozen

=2 spectrum 'B' from Yoshida

=3 spectrum 'C' from Yoshida

=4 National Building Code spectrum.

Note:figures showing the spectrums may be found in the appendix of this manual.

AMAX =maximum ground acceleration as a fraction of gravity.

DAMPIN = the fraction of critical damping that is to be used in the elastic analysis or in the first iteration of the inelastic analysis.

Card 2 TITLE Format(20A4) (one card)
Any appropiate title composed of less than 80 letters, numbers and spaces.

Card 3   NRJ,NRM,E,G   Format(2I5,2F10.0)   (one card)

NRJ =number of joints in the structure

NRM =number of members in the structure

E=Young's modulus

G=Shear modulus. If shear modulus is input as zero, then no shear deflections will be calculated.

Card 4: JN,NDX,NDY,NDR,X,Y   Format(4I5,2F10.3)
(NRJ cards: 1 card/joint)

JN = the node number

NDX =0 if the node cannot move in the x direction
=1 if the node can move in the x direction.
=N if the node is to have the same motion as node N.

NDY =0 if the node cannot move in the y direction
=1 if the node can move in the y direction.
=N if the node is to have the same y motion as node N.

NDR =0 if the node cannot move in the rotation direction
=1 if the node can rotate
=N if the node is to have the same rotation as node N.

Card 5: MN,JNL,JNG,KL,KG,AREA,CRMOM,AV,BMCAP,EXTL,EXTG
FORMAT(5I5,F8.2,F12.3,2F10.3,2F6.3)
(NRM CARDS: 1 CARD/MEMBER)

MN =the member number.

JNL =the lesser joint number

JNG =the greater joint number

Note: 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.

KL =1 if the member is fixed at the lesser joint number.
=0 if the member is pinned at the lesser joint number

KG =1 if the member is fixed at the greater joint number.
=0 if the member is pinned at the greater joint number

AREA =cross sectional area of the member.

CRMOM = the cracked moment of inertia of the member. For elastic analysis the number used here is the inertia desired for actual analysis.

AV =Shear area of the member. At this stage AV should equal zero as testing is incomplete on the program's ability to handle shear deflections.
=0.0 then shear deflections will not be computed.

EXTG =rigid extension on the lesser joint end of member

EXTL =rigid extension on the greater joint end of the member.

NOTE: for proper use of the rigid extensions they must be positive and with a length that has a sum total for both rigid extensions of less than the spacing between the joints which the member is spanning. In other words for the program to execute there cannot be a member with zero or negative elastic length. Note also that at this stage of program development the rigid arms are assumed to be attached only to horizontal members. The attachment of rigid arms to non-horizontal members will result in the printing of an error message.

Card 6. NMASS Format(I5) (one card)

NMASS =The number of nodes to which a weight is attached. This is independant of the number of weights which are attached to those nodes. 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.

Card 7. JN,WTX,WTY,WTR Format(I5,3F10.0)
(NMASS cards: 1 card/joint with mass)

JN = Joint to which the weight is applied.

WTX = Weight in x direction.

WTY = Weight in y direction.

WTR = Rotational weight.

NOTE that weights must be inserted as such. The program converts them to mass by dividing by the standard value for the acceleration of gravity (32.2 ft/sec^2). Also note that once the masses have been assigned to the appropiate degrees of freedom the program does not distinguish between masses opposing motion in the horizontal direction and those opposing motion in the vertical or rotary directions. This means that the same spectral acceleration will be applied to both directions. The user is cautioned that masses opposing motion in directions other than the horizontal direction are in most cases unnecessary. For further details see also 'program restrictions' and 'approximate execution times' in this manual.

OUTPUT

Unit 6

This file is for data from intermediate iterations from inelastic analysis. It should not be needed unless an error occurs, or it is wished to examine the progress of convergence at the conclusion of a run. Nothing of use is written on unit 6 during elastic analysis. The user is cautioned that file 6 may become quite lengthy during runs using the Modified Substitute Structure Method and that it is worthwhile determining firstly if what is on the file is desired and secondly if the file is unreasonably lengthy prior to printout.

UNIT 7

This file contains the majority of the useful output from both elastic and inelastic analysis. Input member data as well as output forces and displacements appear on this file in a manner that should make them reasonably straightforward to understand. Note that for elastic analysis as the input moment capacities have little purpose neither will the output damage ratios. Should the program stop unexpectedly it is inportant that unit 7 be printed out to aid in the debugging process. Unit 7 will contain any error messages that are generated by the error checking routines inside the program itself.

UNIT 8

This file contains the damage ratios for each member at the conclusion of each iteration. It should not be required to be printed after performing elastic analysis.

The user should not assign a file to contain output if he has no interest in ever viewing or printing out that file. For example should elastic analysis be run then file 8 will not be required. Under these circumstances the program will run more efficiently and virtual memory costs will be less if the output file is assigned to *DUMMY*.

OPERATING INSTRUCTIONS

It is assumed for this discussion that the user has a compiled version of EDAM in his file COMPILED and that the desired data file is the file DATA. The following command will run the program.

```
$RUN COMPILED 5=DATA 6=-6 7=-7 8=-8
```

Should elastic analysis be performed then the following command would be a preferable command.

```
$RUN COMPILED 5=DATA 6=*DUMMY* 7=-OUT 8=*DUMMY*
```

LIBRARY SUBROUTINES CALLED

The program calls two main subroutines from the University of British Columbia (UBC) subroutine library. These main subroutines call other routines during their execution. The complete writeup of the programs called can be found in the book UBC MATRIX available from the UBC computing center.

The two main subroutines called are PRITZ and DFBAND both of these subroutines work in extended precision and have themselves undergone rigorus testing before being allowed general access. By using the 'canned' programs EDAM takes advantage of this testing. By calling the subroutines rather than keeping them in source, compilation and storage costs are also saved. Both routines require the stiffness matrix to be input in the same manner, this being the lower half of the matrix including the diagonal to be stored by columns one half-bandwidth after another. In this manner the large doubly subscripted stiffness matrix is stored as a smaller one dimensional array.

PRITZ is an eigenvalue and eigenvector finding routine. At the time of writing it appears to be the best routine publicly available at UBC for this purpose. The program is efficient and also checks that the eigenvalues given are those requested. This means that if the program is analysing ten modes that the eigenvalue finding routine will return with the ten lowest eigenvalues, not the lowest nine and the eleventh. PRITZ prints out a selection of operational information during its execution which includes the number of significant figures to be expected in each eigenvalue and eigenvector and a statement confirming that all those eigenvalues requested have been located as described. The program EDAM supresses this information for all but the last iteration. If the progam is being executed from a terminal screen then this information will appear on the screen shortly before the completion of the run. If the program is being executed from batch, the information will be printed on the same sheet as the execution cost information. The user should note that the manual UBC MATRIX has omitted to inform that the matrix entering PRITZ from which the eigenvalues are to be computed is destroyed during the execution of the subroutine. This problem is circumvented by duplicating the matrix before sending it to the subroutine and retaining the copy which is required later for solving the displacements caused by the forces on the structure. This omission has been pointed out to the appropiate authorities in the computing center and a note should appear in future editions of UBC MATRIX .

DFBAND solves the matrix problem Ax=B where A is a symmetric banded matrix and B is a column matrix. DFBAND was used in EDAM to replace the single precision equivalent FBAND used in the original program MSSM.S. This saves converting the stiffness matrix from double to single precision before solving for the displacements. As an added bonus the execution times listed for DFBAND are less than those for FBAND, though DFBAND offers the option of iterative improvement this is not undertaken in EDAM believing it unnecessary.

TIMING

The timing relationship for the program depends strongly on the following items:

(a) the number of degrees of freedom in the structure.

(b) the halfbandwidth of the structure.

(c) the number of masses attached. (This appears to be an almost direct relationship-doubling the number of input masses will approximatly double the execution cost).

(d) the number of modes included in the analysis.

(e) the number of iterations when doing an MSSM analysis. Examples of execution costs are given in Metten's thesis.

OTHER INSTITUTIONS

The program EDAM has been written to operate on the Michigan Terminal System (MTS) of the University of British Columbia using IBM style fortran and two main UBC canned subroutines. It is expected that transfer to other institutions of this program would involve using the subroutines of that institution to calculate the eigenvalues and solve the standard matrix problem. As these are both problems of frequent occurence it is expected that solution to them will exist at many other institutions. It will be necessary to change the calling command in EDAM to match the subroutine of the institution.

A sample problem is included in the appendix of this manual which should be an aid in determining if the program is executing correctly as well as demonstrating input and output styles.

APPENDIX

Sample problem (Input and ouput)

SAMPLE PROBLEM DATA FILE

```
  30    7    7    1   0.20000   0.05000
TEN STORY TEST WALL TYPE C WALLS 6FT 60 KIP-FT LINTEL
  22   30     3600.       1200.
   1    0    0    0   0.00000   0.00000
   2    0    0    0  21.00000   0.00000
   3    1    1    1   0.00000   7.75000
   4    1    1    1  21.00000   7.75000
   5    1    1    1   0.00000  16.25000
   6    1    1    1  21.00000  16.25000
   7    1    1    1   0.00000  24.75000
   8    1    1    1  21.00000  24.75000
   9    1    1    1   0.00000  33.25000
  10    1    1    1  21.00000  33.25000
  11    1    1    1   0.00000  41.75000
  12    1    1    1  21.00000  41.75000
  13    1    1    1   0.00000  50.25000
  14    1    1    1  21.00000  50.25000
  15    1    1    1   0.00000  58.75000
  16    1    1    1  21.00000  58.75000
  17    1    1    1   0.00000  67.25000
  18    1    1    1  21.00000  67.25000
  19    1    1    1   0.00000  75.75000
  20    1    1    1  21.00000  75.75000
  21    1    1    1   0.00000  84.25000
  22    1    1    1  21.00000  84.25000
   1   21   22    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
   2   19   20    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
   3   17   18    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
   4   15   16    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
   5   13   14    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
   6   11   12    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
   7    9   10    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
   8    7    8    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
   9    5    6    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
  10    3    4    1     1   144.00     1024.000     0.000     60.000  7.50  7.50
  11    1    3    1     1  1620.00  2187000.000     0.000  13000.000  0.00  0.00
  12    2    4    1     1  1620.00  2187000.000     0.000  13000.000  0.00  0.00
  13    3    5    1     1  1620.00  2187000.000     0.000  12666.660  0.00  0.00
  14    4    6    1     1  1620.00  2187000.000     0.000  12666.660  0.00  0.00
  15    5    7    1     1  1620.00  2187000.000     0.000  12333.330  0.00  0.00
  16    6    8    1     1  1620.00  2187000.000     0.000  12333.330  0.00  0.00
  17    7    9    1     1  1620.00  2187000.000     0.000  12000.000  0.00  0.00
  18    8   10    1     1  1620.00  2187000.000     0.000  12000.000  0.00  0.00
  19    9   11    1     1  1620.00  2187000.000     0.000  11666.660  0.00  0.00
  20   10   12    1     1  1620.00  2187000.000     0.000  11666.660  0.00  0.00
  21   11   13    1     1  1620.00  2187000.000     0.000  11333.330  0.00  0.00
  22   12   14    1     1  1620.00  2187000.000     0.000  11333.330  0.00  0.00
  23   13   15    1     1  1620.00  2187000.000     0.000  11000.000  0.00  0.00
  24   14   16    1     1  1620.00  2187000.000     0.000  11000.000  0.00  0.00
  25   15   17    1     1  1620.00  2187000.000     0.000  10666.660  0.00  0.00
```

26	16	18	1	1	1620.00	2187000.000	0.000	10666.660	0.00	0.00
27	17	19	1	1	1620.00	2187000.000	0.000	10333.330	0.00	0.00
28	18	20	1	1	1620.00	2187000.000	0.000	10333.330	0.00	0.00
29	19	21	1	1	1620.00	2187000.000	0.000	10000.000	0.00	0.00
30	20	22	1	1	1620.00	2187000.000	0.000	10000.000	0.00	0.00
10										
4	270.	0.	0.							
6	270.	0.	0.							
8	270.	0.	0.							
10	270.	0.	0.							
12	270.	0.	0.							
14	270.	0.	0.							
16	270.	0.	0.							
18	270.	0.	0.							
20	270.	0.	0.							
22	270.	0.	0.							

SAMPLE PROBLEM
OUTPUT

*******PROGRAM OPTIONS*******

MAXIMUM NUMBER OF MODES IN ANALYSIS 7
INELASTIC ANALYSIS MAXIMUM ITTERATIONS= 30
INITIAL DAMPING RATIO= 0.050
NUMBER OF MODES TO HAVE OUTPUT PRINTED= 7
-SEISMIC INPUT
-MAXIMUM ACCELERATION=0.200 TIMES GRAVITY
SPECTRUM A USED

1TEN STORY TEST WALL TYPE C WALLS 6FT 60 KIP-FT LINTEL
- E = 3600.0 KSI G = 1200.0 KSI

****

-NO. OF JOINTS = 22 NO. OF MEMBERS = 30

-JOINT DATA

JN	X(FEET)	Y(FEET)	NDX	NDY	NDR
1	0.0	0.0	0	0	0
2	21.000	0.0	0	0	0
3	0.0	7.750	1	2	3
4	21.000	7.750	4	5	6
5	0.0	16.250	7	8	9
6	21.000	16.250	10	11	12
7	0.0	24.750	13	14	15
8	21.000	24.750	16	17	18
9	0.0	33.250	19	20	21
10	21.000	33.250	22	23	24
11	0.0	41.750	25	26	27
12	21.000	41.750	28	29	30
13	0.0	50.250	31	32	33
14	21.000	50.250	34	35	36
15	0.0	58.750	37	38	39
16	21.000	58.750	40	41	42
17	0.0	67.250	43	44	45
18	21.000	67.250	46	47	48
19	0.0	75.750	49	50	51
20	21.000	75.750	52	53	54
21	0.0	84.250	55	56	57

22	21.000	84.250	58	59	60

-MEMBER DATA

MN	JNL	JNG	EXTL	LENGTH (FEET)	EXTG	XM(FT)	YM(FT)	AREA (SQ.IN)	I(CRACKED) (IN**4)	AV (SQ.IN)	MOMENT CAPACITY	KL	KG
1	21	22	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
2	19	20	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
3	17	18	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
4	15	16	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
5	13	14	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
6	11	12	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
7	9	10	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
8	7	8	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
9	5	6	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
10	3	4	7.500	6.0000	7.500	6.0000	0.0	144.0	1024.0	0.0	60.00	1	1
11	1	3	0.0	7.7500	0.0	0.0	7.7500	1620.0	2187000.0	0.0	13000.00	1	1
12	2	4	0.0	7.7500	0.0	0.0	7.7500	1620.0	2187000.0	0.0	13000.00	1	1
13	3	5	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	12666.66	1	1
14	4	6	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	12666.66	1	1
15	5	7	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	12333.33	1	1
16	6	8	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	12333.33	1	1
17	7	9	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	12000.00	1	1
18	8	10	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	12000.00	1	1
19	9	11	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	11666.66	1	1
20	10	12	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	11666.66	1	1
21	11	13	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	11333.33	1	1
22	12	14	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	11333.33	1	1
23	13	15	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	11000.00	1	1
24	14	16	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	11000.00	1	1
25	15	17	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	10666.66	1	1
26	16	18	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	10666.66	1	1
27	17	19	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	10333.33	1	1
28	18	20	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	10333.33	1	1
29	19	21	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	10000.00	1	1
30	20	22	0.0	8.5000	0.0	0.0	8.5000	1620.0	2187000.0	0.0	10000.00	1	1

-NO.OF DEGREES OF FREEDOM OF STRUCTURE = 60

HALF BANDWIDTH OF STIFFNESS MATRIX = 9

MEMBER	NP1	NP2	NP3	NP4	NP5	NP6
1	55	56	57	58	59	60
2	49	50	51	52	53	54
3	43	44	45	46	47	48
4	37	38	39	40	41	42
5	31	32	33	34	35	36
6	25	26	27	28	29	30
7	19	20	21	22	23	24
8	13	14	15	16	17	18
9	7	8	9	10	11	12
10	1	2	3	4	5	6
11	0	0	0	1	2	3

12	0	0	0	4	5	6
13	1	2	3	7	8	9
14	4	5	6	10	11	12
15	7	8	9	13	14	15
16	10	11	12	16	17	18
17	13	14	15	19	20	21
18	16	17	18	22	23	24
19	19	20	21	25	26	27
20	22	23	24	28	29	30
21	25	26	27	31	32	33
22	28	29	30	34	35	36
23	31	32	33	37	38	39
24	34	35	36	40	41	42
25	37	38	39	43	44	45
26	40	41	42	46	47	48
27	43	44	45	49	50	51
28	46	47	48	52	53	54
29	49	50	51	55	56	57
30	52	53	54	58	59	60

****

-NO. OF NODES WITH MASS = 10

JN	X-MASS (KIPS)	Y-MASS (KIPS)	ROT.MASS (IN-KIPS)
4	270.000	0.0	0.0
6	270.000	0.0	0.0
8	270.000	0.0	0.0
10	270.000	0.0	0.0
12	270.000	0.0	0.0
14	270.000	0.0	0.0
16	270.000	0.0	0.0
18	270.000	0.0	0.0
20	270.000	0.0	0.0
22	270.000	0.0	0.0

-MASS NO.	DOF	ASSIGNED MASS (KIP*SEC**2/FT)
1	4	8.38509
2	10	8.38509
3	16	8.38509
4	22	8.38509
5	28	8.38509
6	34	8.38509
7	40	8.38509
8	46	8.38509
9	52	8.38509
10	58	8.38509

------INITIAL ELASTIC PERIOD------

MODES	EIGENVALUES	NATURAL FREQUENCIES (RAD/SEC)	(CYCS/SEC)	PERIODS (SECS)	SA (2 PERCENT DAMPING)
1	56.6715	7.5280	1.1981	0.8346	0.3594
2	1141.1099	33.7803	5.3763	0.1860	0.7500
3	6697.1875	81.8363	13.0247	0.0768	0.3839
4	20191.7539	142.0977	22.6157	0.0442	0.2211
5	46299.0742	215.1722	34.2459	0.0292	0.1460
6	93277.6250	305.4138	48.6084	0.0206	0.1029
7	169366.6250	411.5417	65.4993	0.0153	0.0763

INELASTIC RESULTS

-ITERATION NO.	NO. ABOVE CAPACITY	DAMDIF	S MATRIX RATIO
1	10	0.305	0.787E+02
2	10	0.438	0.788E+02
3	10	0.121	0.787E+02
4	0	0.028	0.787E+02
5	0	0.006	0.787E+02

-ITERATION NUMBER 6

ALL ELEMENTS OF MAIN DIAGONAL OF STIFFNESS MATRIX ARE POSITIVE DEFINITE
RATIO OF LARGEST TO SMALLEST DIAGONAL STIFFNESSMATRIX ELEMENT IS 0.787E+02
-NO. OF MODES TO BE ANALIZED = 7

****

****

TOTAL MODE SHAPES CORRESPONDING TO FIRST 7 FREQUENCIES

DOF	1	2	3	4	5	6	7
1	0.014480	-0.089366	0.180759	0.185176	0.136704	-0.091373	-0.058788
2	0.000832	0.000062	0.001622	0.000207	0.001155	0.000118	-0.000499
3	-0.003638	0.021106	-0.039499	-0.036143	-0.022496	0.011360	0.004317
4	0.014500	-0.093449	0.243931	0.434281	0.642261	-0.844956	-0.983011
5	-0.000832	-0.000062	-0.001622	-0.000207	-0.001155	-0.000118	0.000499
6	-0.003644	0.022074	-0.053356	-0.085042	-0.106605	0.107230	0.075470

7	0.060413	-0.323009	0.543219	0.426354	0.212601	-0.073798	-0.006932
8	0.001657	0.000667	0.002252	-0.001070	0.000822	0.001720	-0.000058
9	-0.007052	0.031594	-0.038389	-0.011338	0.010394	-0.015855	-0.011988
10	0.060498	-0.337775	0.733097	1.000000	1.000000	-0.686251	-0.118837
11	-0.001657	-0.000667	-0.002252	0.001070	-0.000822	-0.001720	0.000058
12	-0.007061	0.033039	-0.051814	-0.026563	0.049026	-0.147799	-0.204711
13	0.132796	-0.592181	0.741110	0.314341	-0.012295	0.090194	0.056461
14	0.002391	0.001695	0.002296	-0.002595	0.000878	0.002255	-0.001032
15	-0.009864	0.029747	-0.004390	0.036044	0.033295	-0.011436	0.003732
16	0.132983	-0.619245	1.000000	0.736926	-0.055688	0.829688	0.948003
17	-0.002391	-0.001695	-0.002296	0.002595	-0.000878	-0.002255	0.001032
18	-0.009878	0.031107	-0.005905	0.084618	0.156614	-0.106476	0.062653
19	0.226589	-0.800561	0.595616	-0.092702	-0.207787	0.050825	-0.040850
20	0.003034	0.003013	0.002293	-0.003550	0.001837	0.002232	-0.001692
21	-0.012092	0.017805	0.037481	0.050040	0.004301	0.017922	0.008860
22	0.226908	-0.837141	0.803557	-0.217068	-0.972834	0.471274	-0.673248
23	-0.003034	-0.003013	-0.002293	0.003550	-0.001837	-0.002232	0.001692
24	-0.012110	0.018618	0.050594	0.117323	0.020299	0.165483	0.149593
25	0.336946	-0.876319	0.155338	-0.394789	-0.066773	-0.102618	-0.024142
26	0.003584	0.004476	0.002637	-0.003852	0.002893	0.002929	-0.001733
27	-0.013768	-0.000762	0.061014	0.013716	-0.031325	0.007404	-0.011063
28	0.337421	-0.916357	0.209492	-0.924509	-0.313806	-0.939801	-0.400681
29	-0.003584	-0.004476	-0.002637	0.003852	-0.002893	-0.002929	0.001733
30	-0.013787	-0.000797	0.082336	0.032111	-0.146764	0.068657	-0.185375
31	0.459345	-0.780754	-0.348535	-0.287772	0.182996	-0.028460	0.060082
32	0.004042	0.005937	0.003477	-0.003998	0.003336	0.003890	-0.002453
33	-0.014935	-0.021745	0.051414	-0.036140	-0.016588	-0.019700	0.000139
34	0.459992	-0.816428	-0.470298	-0.673617	0.853931	-0.263258	1.000000
35	-0.004042	-0.005937	-0.003477	0.003998	-0.003336	-0.003890	0.002453
36	-0.014956	-0.022738	0.069378	-0.084719	-0.077633	-0.181016	0.001738
37	0.589724	-0.511407	-0.636027	0.112152	0.140570	0.109676	-0.025701
38	0.004406	0.007259	0.004704	-0.004531	0.003482	0.004216	-0.003169
39	-0.015658	-0.040973	0.012418	-0.047861	0.024660	-0.003182	0.011053
40	0.590555	-0.534783	-0.858189	0.263109	0.654825	1.000000	-0.416777
41	-0.004406	-0.007259	-0.004704	0.004531	-0.003482	-0.004216	0.003169
42	-0.015680	-0.042845	0.016765	-0.112175	0.115596	-0.028864	0.184100
43	0.724649	-0.097985	-0.531185	0.380906	-0.122555	0.006524	-0.040948
44	0.004679	0.008334	0.006022	-0.005548	0.004052	0.004448	-0.003298
45	-0.016020	-0.055200	-0.036672	-0.008149	0.025556	0.020131	-0.009016
46	0.725670	-0.102482	-0.716832	0.892665	-0.575976	0.051683	-0.664422
47	-0.004679	-0.008334	-0.006022	0.005548	-0.004052	-0.004448	0.003298
48	-0.016043	-0.057723	-0.049475	-0.019113	0.119679	0.185371	-0.151033
49	0.861487	0.409079	-0.048502	0.211016	-0.170379	-0.103605	0.055257
50	0.004861	0.009082	0.007091	-0.006649	0.005050	0.005284	-0.003898
51	-0.016127	-0.062921	-0.072953	0.045611	-0.017371	-0.002399	-0.003307
52	0.862701	0.427760	-0.065562	0.494502	-0.797822	-0.953821	0.924039
53	-0.004861	-0.009082	-0.007091	0.006649	-0.005050	-0.005284	0.003898
54	-0.016150	-0.065801	-0.098475	0.107006	-0.081857	-0.023383	-0.052926
55	0.998592	0.956252	0.641494	-0.307975	0.124753	0.049196	-0.018785
56	0.004952	0.009462	0.007666	-0.007311	0.005763	0.006049	-0.004663

57	-0.016106	-0.064993	-0.085127	0.068598	-0.043200	-0.025556	0.014511
58	1.000000	1.000000	0.865994	-0.723500	0.590963	0.469507	-0.345542
59	-0.004952	-0.009462	-0.007666	0.007311	-0.005763	-0.006049	0.004663
60	-0.016129	-0.067976	-0.114993	0.161251	-0.203948	-0.239265	0.250288

MODES	EIGENVALUES	NATURAL FREQUENCIES (RAD/SEC)	(CYCS/SEC)	PERIODS (SECS)	SA (2 PERCENT DAMPING)
1	29.0268	5.3877	0.8575	1.1662	0.2572
2	920.2505	30.3356	4.8281	0.2071	0.7500
3	6249.0859	79.0512	12.5815	0.0795	0.3974
4	19637.9102	140.1353	22.3033	0.0448	0.2242
5	45667.3555	213.6992	34.0115	0.0294	0.1470
6	92555.0000	304.2285	48.4197	0.0207	0.1033
7	168589.8125	410.5969	65.3489	0.0153	0.0765

MASS MODE SHAPES CORRESPONDING TO FIRST 7 FREQUENCIES

MASS	1	2	3	4	5	6	7
1	0.014500	-0.093449	0.243931	0.434281	0.642261	-0.844956	-0.983011
2	0.060498	-0.337775	0.733097	1.000000	1.000000	-0.686251	-0.118837
3	0.132983	-0.619245	1.000000	0.736926	-0.055688	0.829688	0.948003
4	0.226908	-0.837141	0.803557	-0.217068	-0.972834	0.471274	-0.673248
5	0.337421	-0.916357	0.209492	-0.924509	-0.313806	-0.939801	-0.400681
6	0.459992	-0.816428	-0.470298	-0.673617	0.853931	-0.263258	1.000000
7	0.590555	-0.534783	-0.858189	0.263109	0.654825	1.000000	-0.416777
8	0.725670	-0.102482	-0.716832	0.892665	-0.575976	0.051683	-0.664422
9	0.862701	0.427760	-0.065562	0.494502	-0.797822	-0.953821	0.924039
10	1.000000	1.000000	0.865994	-0.723500	0.590963	0.469507	-0.345542

-MODAL PARTICIPATION FACTOR

MODE	1	1.46159
MODE	2	-0.67494
MODE	3	0.38676
MODE	4	0.27170
MODE	5	0.20783
MODE	6	-0.16714
MODE	7	-0.14376

MODE 1 CONTRIBUTION FACTOR= 0.28314
MODE 2 CONTRIBUTION FACTOR=-0.47085
MODE 3 CONTRIBUTION FACTOR= 0.15008
MODE 4 CONTRIBUTION FACTOR= 0.06028
MODE 5 CONTRIBUTION FACTOR= 0.03040
MODE 6 CONTRIBUTION FACTOR=-0.01721
MODE 7 CONTRIBUTION FACTOR=-0.01098

-MODE	SMEARED DAMPING RATIO
1	0.04618
2	0.02599
3	0.02192
4	0.02083
5	0.02040

6	0.02021
7	0.02012

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MODE NUMBER 1 MODAL FORCES AND DISPLACEMENTS

JOINT NO.	X-DISP(FT)	Y-DISP(FT)	ROTATION(RAD)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.0046	0.0003	-0.0011
4	0.0046	-0.0003	-0.0011
5	0.0190	0.0005	-0.0022
6	0.0190	-0.0005	-0.0022
7	0.0417	0.0008	-0.0031
8	0.0418	-0.0008	-0.0031
9	0.0712	0.0010	-0.0038
10	0.0713	-0.0010	-0.0038
11	0.1059	0.0011	-0.0043
12	0.1060	-0.0011	-0.0043
13	0.1444	0.0013	-0.0047
14	0.1446	-0.0013	-0.0047
15	0.1853	0.0014	-0.0049
16	0.1856	-0.0014	-0.0049
17	0.2277	0.0015	-0.0050
18	0.2280	-0.0015	-0.0050
19	0.2707	0.0015	-0.0051
20	0.2711	-0.0015	-0.0051
21	0.3138	0.0016	-0.0051
22	0.3143	-0.0016	-0.0051

MN	AXIAL KIPS	SHEAR KIPS	BML (K-FT)	BMG (K-FT)
1	38.232	-19.558	58.669	-58.677
2	32.959	-19.588	58.759	-58.767
3	27.740	-19.683	59.045	-59.053
4	22.577	-19.827	59.477	-59.485
5	17.582	-19.963	59.884	-59.892
6	12.895	-20.022	60.061	-60.069
7	8.672	-19.954	59.857	-59.865
8	5.082	-19.748	59.240	-59.248
9	2.311	-19.434	58.299	-58.307
10	0.553	-19.064	57.188	-57.196
11	196.839	168.589	-8719.730	-7413.168
12	-196.839	168.795	-8731.918	-7423.758
13	177.775	168.030	-7613.270	-6185.016
14	-177.775	168.239	-7623.887	-6193.852
15	158.341	165.712	-6388.941	-4980.383
16	-158.341	165.915	-6397.809	-4987.527
17	138.593	160.604	-5187.566	-3822.430
18	-138.593	160.824	-5194.801	-3827.795
19	118.639	151.961	-4031.718	-2740.051
20	-118.639	152.130	-4037.229	-2744.124

21	98.618	139.021	-2950.104	-1768.424
22	-98.618	139.224	-2954.274	-1770.868
23	78.655	121.464	-1977.663	-945.220
24	-78.655	121.667	-1980.538	-946.370
25	58.828	98.967	-1153.405	-312.182
26	-58.828	99.035	-1154.747	-312.949
27	39.145	71.194	-519.121	86.025
28	-39.145	71.244	-519.648	85.929
29	19.557	38.193	-119.428	205.214
30	-19.557	38.227	-119.716	205.213

MODE 1 CONTRIBUTION FACTOR= 0.28328
DAMPING=0.0462 PERIOD=1.1662 SEC. SA=0.194
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MODE NUMBER 2 MODAL FORCES AND DISPLACEMENTS

JOINT NO.	X-DISP(FT)	Y-DISP(FT)	ROTATION(RAD)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.0015	-0.0000	-0.0003
4	0.0015	0.0000	-0.0004
5	0.0053	-0.0000	-0.0005
6	0.0056	0.0000	-0.0005
7	0.0098	-0.0000	-0.0005
8	0.0102	0.0000	-0.0005
9	0.0132	-0.0000	-0.0003
10	0.0138	0.0000	-0.0003
11	0.0144	-0.0001	0.0000
12	0.0151	0.0001	0.0000
13	0.0129	-0.0001	0.0004
14	0.0135	0.0001	0.0004
15	0.0084	-0.0001	0.0007
16	0.0088	0.0001	0.0007
17	0.0016	-0.0001	0.0009
18	0.0017	0.0001	0.0010
19	-0.0067	-0.0001	0.0010
20	-0.0070	0.0001	0.0011
21	-0.0158	-0.0002	0.0011
22	-0.0165	0.0002	0.0011

MN	AXIAL KIPS	SHEAR KIPS	BML (K-FT)	BMG (K-FT)
1	-62.288	4.299	-12.869	12.925
2	-26.597	4.161	-12.455	12.509
3	6.403	3.687	-11.036	11.083
4	33.282	2.810	-8.413	8.449
5	50.791	1.558	-4.665	4.685
6	57.005	0.028	-0.082	0.083
7	52.082	-1.639	4.906	-4.926
8	38.533	-3.285	9.834	-9.875
9	21.022	-4.783	14.319	-14.380
10	5.813	-6.061	18.145	-18.223

11	-0.774	176.046	-3135.883	-1771.528
12	0.774	183.794	-3278.397	-1853.993
13	-6.836	170.230	-1835.135	-388.182
14	6.836	177.717	-1917.637	-407.046
15	-11.619	149.205	-438.341	829.901
16	11.619	155.797	-457.265	867.013
17	-14.904	110.678	795.446	1736.206
18	14.904	115.590	832.495	1815.012
19	-16.542	58.593	1719.019	2217.062
20	16.542	61.222	1797.786	2318.177
21	-16.515	1.593	2217.358	2230.900
22	16.515	1.704	2318.474	2332.959
23	-14.956	-49.200	2247.261	1829.058
24	14.956	-51.315	2349.343	1913.168
25	-12.146	-82.479	1858.549	1157.479
26	12.146	-86.031	1942.669	1211.406
27	-8.459	-88.872	1196.115	440.707
28	8.459	-92.652	1250.075	462.534
29	-4.299	-62.277	484.286	-45.073
30	4.299	-64.858	506.166	-45.127

MODE 2 CONTRIBUTION FACTOR=-0.47095
DAMPING=0.0260 PERIOD=0.2071 SEC. SA=0.698
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MODE NUMBER 3 MODAL FORCES AND DISPLACEMENTS

JOINT NO.	X-DISP(FT)	Y-DISP(FT)	ROTATION(RAD)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.0001	0.0000	-0.0000
4	0.0002	-0.0000	-0.0000
5	0.0004	0.0000	-0.0000
6	0.0006	-0.0000	-0.0000
7	0.0006	0.0000	-0.0000
8	0.0008	-0.0000	-0.0000
9	0.0005	0.0000	0.0000
10	0.0006	-0.0000	0.0000
11	0.0001	0.0000	0.0000
12	0.0002	-0.0000	0.0001
13	-0.0003	0.0000	0.0000
14	-0.0004	-0.0000	0.0001
15	-0.0005	0.0000	0.0000
16	-0.0007	-0.0000	0.0000
17	-0.0004	0.0000	-0.0000
18	-0.0006	-0.0000	-0.0000
19	-0.0000	0.0000	-0.0001
20	-0.0001	-0.0000	-0.0001
21	0.0005	0.0000	-0.0001
22	0.0007	-0.0000	-0.0001

MN	AXIAL KIPS	SHEAR KIPS	BML (K-FT)	BMG (K-FT)

1	15.001	-0.306	0.904	-0.930
2	-1.140	-0.262	0.773	-0.796
3	-12.405	-0.132	0.391	-0.402
4	-14.845	0.048	-0.142	0.146
5	-8.136	0.205	-0.606	0.623
6	3.619	0.264	-0.780	0.803
7	13.895	0.184	-0.544	0.560
8	17.299	-0.025	0.073	-0.076
9	12.688	-0.311	0.920	-0.947
10	4.221	-0.610	1.803	-1.855
11	0.944	30.197	-332.524	-98.500
12	-0.944	40.527	-448.161	-134.076
13	0.334	25.975	-104.872	115.916
14	-0.334	34.863	-140.499	155.834
15	0.023	13.287	112.665	225.607
16	-0.023	17.842	152.556	304.214
17	-0.002	-4.012	225.347	191.249
18	0.002	-5.383	303.951	258.192
19	0.182	-17.906	193.170	40.970
20	-0.182	-24.053	260.130	55.683
21	0.446	-21.525	43.725	-139.237
22	-0.446	-28.923	58.461	-187.389
23	0.651	-13.389	-137.096	-250.901
24	-0.651	-18.001	-185.231	-338.243
25	0.699	1.456	-250.397	-238.020
26	-0.699	1.932	-337.735	-321.317
27	0.567	13.861	-239.400	-121.583
28	-0.567	18.576	-322.707	-164.815
29	0.306	15.001	-124.313	3.193
30	-0.306	20.092	-167.566	3.220

MODE 3 CONTRIBUTION FACTOR= 0.15009
DAMPING=0.0219 PERIOD=0.0795 SEC. SA=0.388

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MODE NUMBER 4 MODAL FORCES AND DISPLACEMENTS

JOINT NO.	X-DISP(FT)	Y-DISP(FT)	ROTATION(RAD)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.0000	0.0000	-0.0000
4	0.0000	-0.0000	-0.0000
5	0.0000	-0.0000	-0.0000
6	0.0001	0.0000	-0.0000
7	0.0000	-0.0000	0.0000
8	0.0001	0.0000	0.0000
9	-0.0000	-0.0000	0.0000
10	-0.0000	0.0000	0.0000
11	-0.0000	-0.0000	0.0000
12	-0.0001	0.0000	0.0000
13	-0.0000	-0.0000	-0.0000
14	-0.0001	0.0000	-0.0000
15	0.0000	-0.0000	-0.0000

16	0.0000	0.0000	-0.0000
17	0.0000	-0.0000	-0.0000
18	0.0001	0.0000	-0.0000
19	0.0000	-0.0000	0.0000
20	0.0000	0.0000	0.0000
21	-0.0000	-0.0000	0.0000
22	-0.0001	0.0000	0.0000

MN	AXIAL KIPS	SHEAR KIPS	BML (K-FT)	BMG (K-FT)
1	-3.549	0.045	-0.130	0.140
2	2.421	0.030	-0.086	0.093
3	4.371	-0.006	0.016	-0.017
4	1.289	-0.033	0.095	-0.102
5	-3.295	-0.026	0.076	-0.082
6	-4.524	0.011	-0.030	0.033
7	-1.062	0.044	-0.128	0.138
8	3.609	0.039	-0.112	0.121
9	4.899	-0.017	0.049	-0.053
10	2.127	-0.102	0.294	-0.318
11	0.015	6.286	-49.564	-0.845
12	-0.015	14.593	-115.851	-2.756
13	-0.087	4.159	-1.904	33.447
14	0.087	9.652	-3.838	78.202
15	-0.103	-0.740	33.272	26.980
16	0.103	-1.726	78.023	63.356
17	-0.065	-4.349	27.382	-9.585
18	0.065	-10.111	63.766	-22.178
19	-0.020	-3.287	-9.126	-37.065
20	0.020	-7.640	-21.708	-86.649
21	-0.010	1.237	-36.955	-26.442
22	0.010	2.884	-86.537	-62.026
23	-0.036	4.532	-26.714	11.809
24	0.036	10.552	-62.304	27.392
25	-0.069	3.243	11.467	39.032
26	0.069	7.559	27.043	91.296
27	-0.075	-1.128	38.973	29.389
28	0.075	-2.600	91.236	69.140
29	-0.045	-3.549	29.697	-0.466
30	0.045	-8.227	69.455	-0.477

MODE 4 CONTRIBUTION FACTOR= 0.06028
DAMPING=0.0208 PERIOD=0.0448 SEC. SA=0.222

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MODE NUMBER 5 MODAL FORCES AND DISPLACEMENTS

JOINT NO.	X-DISP(FT)	Y-DISP(FT)	ROTATION(RAD)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.0000	0.0000	-0.0000
4	0.0000	-0.0000	-0.0000
5	0.0000	0.0000	0.0000

6	0.0000	-0.0000	0.0000
7	-0.0000	0.0000	0.0000
8	-0.0000	-0.0000	0.0000
9	-0.0000	0.0000	0.0000
10	-0.0000	-0.0000	0.0000
11	-0.0000	0.0000	-0.0000
12	-0.0000	-0.0000	-0.0000
13	0.0000	0.0000	-0.0000
14	0.0000	-0.0000	-0.0000
15	0.0000	0.0000	0.0000
16	0.0000	-0.0000	0.0000
17	-0.0000	0.0000	0.0000
18	-0.0000	-0.0000	0.0000
19	-0.0000	0.0000	-0.0000
20	-0.0000	-0.0000	-0.0000
21	0.0000	0.0000	-0.0000
22	0.0000	-0.0000	-0.0000

MN	AXIAL KIPS	SHEAR KIPS	BML (K-FT)	BMG (K-FT)
1	0.863	-0.010	0.030	-0.033
2	-1.162	-0.004	0.012	-0.013
3	-0.840	0.006	-0.018	0.020
4	0.952	0.006	-0.018	0.020
5	1.243	-0.004	0.012	-0.014
6	-0.458	-0.009	0.025	-0.029
7	-1.417	0.001	-0.004	0.005
8	-0.080	0.013	-0.037	0.042
9	1.458	0.006	-0.016	0.018
10	0.936	-0.024	0.066	-0.075
11	0.019	1.497	-9.201	2.397
12	-0.019	6.924	-42.954	10.710
13	-0.005	0.560	2.154	6.916
14	0.005	2.589	10.458	32.461
15	0.001	-0.898	6.975	-0.659
16	-0.001	-4.162	32.522	-2.852
17	0.014	-0.818	-0.522	-7.474
18	-0.014	-3.785	-2.710	-34.882
19	0.016	0.599	-7.459	-2.366
20	-0.016	2.784	-34.867	-11.205
21	0.007	1.057	-2.459	6.523
22	-0.007	4.902	-11.302	30.367
23	0.002	-0.186	6.478	4.897
24	-0.002	-0.865	30.320	22.969
25	0.008	-1.138	4.962	-4.715
26	-0.008	-5.288	23.035	-21.909
27	0.015	-0.299	-4.650	-7.188
28	-0.015	-1.399	-21.842	-33.737
29	0.010	0.863	-7.231	0.108
30	-0.010	3.987	-33.782	0.112

MODE 5 CONTRIBUTION FACTOR= 0.03040
DAMPING=0.0204 PERIOD=0.0294 SEC. SA=0.146

-**

MODE NUMBER 6 MODAL FORCES AND DISPLACEMENTS

JOINT NO.	X-DISP(FT)	Y-DISP(FT)	ROTATION(RAD)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.0000	-0.0000	-0.0000
4	0.0000	0.0000	-0.0000
5	0.0000	-0.0000	0.0000
6	0.0000	0.0000	0.0000
7	-0.0000	-0.0000	0.0000
8	-0.0000	0.0000	0.0000
9	-0.0000	-0.0000	-0.0000
10	-0.0000	0.0000	-0.0000
11	0.0000	-0.0000	-0.0000
12	0.0000	0.0000	-0.0000
13	0.0000	-0.0000	0.0000
14	0.0000	0.0000	0.0000
15	-0.0000	-0.0000	0.0000
16	-0.0000	0.0000	0.0000
17	-0.0000	-0.0000	-0.0000
18	-0.0000	0.0000	-0.0000
19	0.0000	-0.0000	0.0000
20	0.0000	0.0000	0.0000
21	-0.0000	-0.0000	0.0000
22	-0.0000	0.0000	0.0000

MN	AXIAL KIPS	SHEAR KIPS	BML (K-FT)	BMG (K-FT)
1	-0.217	0.003	-0.009	0.010
2	0.440	0.000	-0.001	0.001
3	-0.023	-0.002	0.007	-0.008
4	-0.461	0.000	-0.001	0.001
5	0.121	0.003	-0.007	0.008
6	0.433	-0.001	0.003	-0.004
7	-0.218	-0.003	0.008	-0.010
8	-0.383	0.002	-0.006	0.007
9	0.317	0.004	-0.012	0.014
10	0.390	-0.006	0.017	-0.020
11	-0.001	0.400	-2.029	1.069
12	0.001	3.625	-18.577	9.516
13	-0.007	0.010	1.007	1.090
14	0.007	0.088	9.451	10.197
15	-0.002	-0.307	1.135	-1.475
16	0.002	-2.785	10.244	-13.428
17	0.000	0.076	-1.452	-0.810
18	-0.000	0.689	-13.403	-7.550
19	-0.003	0.293	-0.840	1.651
20	0.003	2.661	-7.581	15.041

21	-0.004	-0.140	1.640	0.449
22	0.004	-1.273	15.030	4.206
23	-0.001	-0.262	0.475	-1.748
24	0.001	-2.375	4.234	-15.957
25	-0.001	0.199	-1.744	-0.052
26	0.001	1.812	-15.952	-0.553
27	-0.003	0.222	-0.077	1.813
28	0.003	2.029	-0.580	16.663
29	-0.003	-0.217	1.816	-0.032
30	0.003	-1.965	16.666	-0.034

MODE 6 CONTRIBUTION FACTOR=-0.01721
DAMPING=0.0202 PERIOD=0.0207 SEC. SA=0.103
-***

MODE NUMBER 7 MODAL FORCES AND DISPLACEMENTS

JOINT NO.	X-DISP(FT)	Y-DISP(FT)	ROTATION(RAD)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.0000	0.0000	-0.0000
4	0.0000	-0.0000	-0.0000
5	0.0000	0.0000	0.0000
6	0.0000	-0.0000	0.0000
7	-0.0000	0.0000	-0.0000
8	-0.0000	-0.0000	-0.0000
9	0.0000	0.0000	-0.0000
10	0.0000	-0.0000	-0.0000
11	0.0000	0.0000	0.0000
12	0.0000	-0.0000	0.0000
13	-0.0000	0.0000	-0.0000
14	-0.0000	-0.0000	-0.0000
15	0.0000	0.0000	-0.0000
16	0.0000	-0.0000	-0.0000
17	0.0000	0.0000	0.0000
18	0.0000	-0.0000	0.0000
19	-0.0000	0.0000	0.0000
20	-0.0000	-0.0000	0.0000
21	0.0000	0.0000	-0.0000
22	0.0000	-0.0000	-0.0000

MN	AXIAL KIPS	SHEAR KIPS	BML (K-FT)	BMG (K-FT)
1	0.059	-0.001	0.003	-0.004
2	-0.157	0.000	-0.001	0.001
3	0.113	0.001	-0.002	0.002
4	0.071	-0.001	0.002	-0.003
5	-0.170	-0.000	0.000	-0.000
6	0.068	0.001	-0.003	0.003
7	0.115	-0.001	0.002	-0.003
8	-0.162	-0.000	0.001	-0.001
9	0.020	0.002	-0.006	0.007
10	0.168	-0.001	0.004	-0.005

11	0.001	0.124	-0.546	0.418
12	-0.001	2.042	-9.028	6.795
13	-0.001	-0.043	0.403	0.037
14	0.001	-0.706	6.780	0.781
15	0.001	-0.063	0.058	-0.482
16	-0.001	-1.038	0.803	-8.018
17	0.001	0.098	-0.486	0.348
18	-0.001	1.612	-8.023	5.677
19	0.000	-0.016	0.339	0.199
20	-0.000	-0.270	5.667	3.372
21	0.001	-0.085	0.209	-0.511
22	-0.001	-1.390	3.382	-8.432
23	0.001	0.086	-0.511	0.217
24	-0.001	1.405	-8.432	3.511
25	0.000	0.015	0.208	0.334
26	-0.000	0.240	3.502	5.542
27	0.001	-0.098	0.340	-0.495
28	-0.001	-1.617	5.549	-8.197
29	0.001	0.059	-0.492	0.011
30.	-0.001	0.965	-8.194	0.012

MODE 7 CONTRIBUTION FACTOR=-0.01098
DAMPING=0.0201 PERIOD=0.0153 SEC. SA=0.076

-***

-ROOT MEAN SQUARE DISPLACEMENTS

JOINT NO.	X-DISP(FT)	Y-DISP(FT)	ROTATION(RAD)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.0048	0.0003	0.0012
4	0.0048	0.0003	0.0012
5	0.0197	0.0005	0.0023
6	0.0198	0.0005	0.0023
7	0.0429	0.0008	0.0031
8	0.0430	0.0008	0.0031
9	0.0724	0.0010	0.0038
10	0.0726	0.0010	0.0038
11	0.1069	0.0011	0.0043
12	0.1071	0.0011	0.0043
13	0.1449	0.0013	0.0047
14	0.1452	0.0013	0.0047
15	0.1855	0.0014	0.0050
16	0.1858	0.0014	0.0050
17	0.2277	0.0015	0.0051
18	0.2281	0.0015	0.0051
19	0.2708	0.0015	0.0052
20	0.2712	0.0015	0.0052
21	0.3142	0.0016	0.0052
22	0.3147	0.0016	0.0052

-ROOT MEAN SQUARE FORCES
0 RSS BASE SHEAR = 498.853 KIPS

MN	AXIAL	SHEAR	BML	BMG	MOMENT

MN	AXIAL KIPS	SHEAR KIPS	BML (K-FT)	BMG (K-FT)
1	0.059	-0.001	0.003	-0.004
2	-0.157	0.000	-0.001	0.001
3	0.113	0.001	-0.002	0.002
4	0.071	-0.001	0.002	-0.003
5	-0.170	-0.000	0.000	-0.000
6	0.068	0.001	-0.003	0.003
7	0.115	-0.001	0.002	-0.003
8	-0.162	-0.000	0.001	-0.001
9	0.020	0.002	-0.006	0.007
10	0.168	-0.001	0.004	-0.005
11	0.001	0.124	-0.546	0.418
12	-0.001	2.042	-9.028	6.795
13	-0.001	-0.043	0.403	0.037
14	0.001	-0.706	6.780	0.781
15	0.001	-0.063	0.058	-0.482
16	-0.001	-1.038	0.803	-8.018
17	0.001	0.098	-0.486	0.348
18	-0.001	1.612	-8.023	5.677
19	0.000	-0.016	0.339	0.199
20	-0.000	-0.270	5.667	3.372
21	0.001	-0.085	0.209	-0.511
22	-0.001	-1.390	3.382	-8.432
23	0.001	0.086	-0.511	0.217
24	-0.001	1.405	-8.432	3.511
25	0.000	0.015	0.208	0.334
26	-0.000	0.240	3.502	5.542
27	0.001	-0.098	0.340	-0.495
28	-0.001	-1.617	5.549	-8.197
29	0.001	0.059	-0.492	0.011
30	-0.001	0.965	-8.194	0.012

MODE 7 CONTRIBUTION FACTOR=-0.01098
DAMPING=0.0201 PERIOD=0.0153 SEC. SA=0.076

-***

-ROOT MEAN SQUARE DISPLACEMENTS

JOINT NO.	X-DISP(FT)	Y-DISP(FT)	ROTATION(RAD)
1	0.0	0.0	0.0
2	0.0	0.0	0.0
3	0.0048	0.0003	0.0012
4	0.0048	0.0003	0.0012
5	0.0197	0.0005	0.0023
6	0.0198	0.0005	0.0023
7	0.0429	0.0008	0.0031
8	0.0430	0.0008	0.0031
9	0.0724	0.0010	0.0038
10	0.0726	0.0010	0.0038
11	0.1069	0.0011	0.0043
12	0.1071	0.0011	0.0043
13	0.1449	0.0013	0.0047
14	0.1452	0.0013	0.0047
15	0.1855	0.0014	0.0050
16	0.1858	0.0014	0.0050
17	0.2277	0.0015	0.0051
18	0.2281	0.0015	0.0051
19	0.2708	0.0015	0.0052
20	0.2712	0.0015	0.0052
21	0.3142	0.0016	0.0052
22	0.3147	0.0016	0.0052

-ROOT MEAN SQUARE FORCES

	KIPS	KIPS	(K-FT)	(K-FT)	CAPACITY	RATIO
1	74.699	20.027	60.070	60.091	60.000	7.520
2	42.455	20.026	60.069	60.089	60.000	7.522
3	31.373	20.026	60.069	60.086	60.000	7.443
4	42.902	20.025	60.069	60.082	60.000	7.228
5	54.475	20.024	60.068	60.078	60.000	6.855
6	58.735	20.023	60.066	60.074	60.000	6.306
7	54.626	20.022	60.060	60.070	60.000	5.563
8	42.697	20.019	60.051	60.066	60.000	4.588
9	25.189	20.017	60.039	60.062	60.000	3.335
10	7.583	20.014	60.025	60.058	60.000	1.755
11	196.843	245.699	9272.570	7622.535	13000.000	0.713
12	196.843	253.363	9338.672	7652.949	13000.000	0.718
13	177.907	240.634	7832.023	6198.359	12666.660	0.618
14	177.907	247.393	7862.629	6209.750	12666.660	0.621
15	158.767	223.385	6405.039	5054.160	12333.328	0.519
16	158.767	228.360	6416.504	5071.875	12333.328	0.520
17	139.392	195.138	5253.102	4202.629	12000.000	0.438
18	139.392	198.429	5270.266	4244.375	12000.000	0.439
19	119.787	163.882	4387.164	3525.095	11666.660	0.376
20	119.787	165.962	4427.262	3593.762	11666.660	0.379
21	99.992	140.696	3690.945	2850.327	11333.328	0.326
22	99.992	142.333	3756.903	2935.752	11333.328	0.331
23	80.067	131.811	2996.812	2074.128	11000.000	0.272
24	80.067	133.715	3079.146	2161.430	11000.000	0.280
25	60.073	128.885	2201.682	1222.870	10666.660	0.206
26	60.073	131.535	2285.387	1295.195	10666.660	0.214
27	40.053	114.718	1326.283	466.180	10333.328	0.128
28	40.053	118.409	1394.880	504.726	10333.328	0.135
29	20.027	74.670	514.963	210.130	10000.000	0.051
30	20.027	78.485	552.199	210.142	10000.000	0.055
6	0	0.002	0.787E+02			

\- NO. OF ITERATIONS = 6

\- BETA=0.0

BENDING MOMENT ERROR=0.050000

DAMAGE RATIO ERROR= 0.010

PROGRAM LISTING

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C=============================================================
C=============================================================
C-------------------------------------------------------------
C     MODAL ANALYSIS PROGRAM 'EDAM' MARCH 1981
C     (ELASTIC AND/OR DAMAGE AFFECTED MODAL ANALYSIS)
C     PROGRAM ORIGINALY WRITTEN BY SUMIO YOSHIDA TITLED MSSM
C     EXTENSIVELY REWRITTEN AND EXPANDED BY ANDREW W.F. METTEN
C-------------------------------------------------------------
C=============================================================
C
C     DOUBLE PRECISION STIFFNESS MATRIX ROUTINE
      REAL*8 S(2000)
      DIMENSION KL(100),KG(100),AREA(100),CRMOM(100),BMCAP(100),
     1          DAMRAT(100),ND(3,100),NP(6,100),XM(100),YM(100),DM(100),
     2          F(300),EXTL(100),EXTG(100),TITLE(20),SDAMP(100),AV(100)
      DIMENSION  DAMB(100), MDOF(50)
      DIMENSION AMASS(300),EVAL(10),EVEC(50,10)
      DIMENSION BMY(75), BETAM(10)
C  PROGRAM DIMENSIONED FOR A MAXIMUM OF
C      100 MEMBERS
C      100 JOINTS
C      50 ASSIGNED MASSES
C      10 EIGENVALUES
C      300 UNKNOWNS
C      (NUMBER OF UNKNOWNS)*(HALF BANDWIDTH) IS LESS THAN 2000
C
C      IUNIT DEFINES THE INPUT AND OUTPUT FILES
C      IUNIT=5 IS DATA SOURCE FILE
C      IUNIT=6 IS TEMPORARY STORAGE FOR INTERMEDIATE DATA
C      IUNIT=7 IS FINAL OUTPUT FILE
C      IUNIT=8 IS DAMAGE RATIO FILE THIS IS SEPARATE FROM OTHER FINAL
C              OUTPUT FILE TO MAKE PLOTTING OF RESULTS EASIER.
       IUNIT=7
C      SUBROUTINE CONTRL READS IN DATA SUCH AS THE NUMBER OF JOINTS
C      AND THE TITLE OF THE STRUCTURE, AND PROGRAM OPTIONS.
C      SUBROUTINE CONTRL IS INDEXED FROM 1001
C
       CALL CONTRL(TITLE,NRJ,NRM,E,G,7,AMAX,ISPEC,DAMPIN,
     1      INELAS,NMODES,NPRINT)
C
C  IDIM DIMENSIONS STRUCTURE AND MATRICES FOR SUBROUTINES
       IDIM=2000
C      SUBROUTINE SETUP READS AND ECHO PRINTS THE MEMBER AND JOINT DATA
C      ITEMS SUCH AS HALF BANDWIDTH AND NUMBER OF UNKNOWNS ARE CALCULATED
C      SUBROUTINE SETUP IS INDEXED FROM 2001.
C
       IFLAG=0
       CALL SETUP(NRJ,NRM,E,G,XM,YM,DM,ND,NP,AREA,CRMOM,DAMRAT,AV,KL,KG,
     1      NU,NB,SDAMP,BMCAP,IUNIT,EXTL,EXTG)
C
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C  CHECK IF IDIM HAS BEEN ASSIGNED LARGE ENOUGH
C  LSTM=LENGTH OF STIFFNESS MATRIX
      LSTM=NU*NB
      IF(LSTM.GT.IDIM) WRITE(7,10) LSTM,IDIM
10    FORMAT(/// 'PROGRAM STOPPED',//'LENGTH OF STIFFNESS MATRIX=',
     1       I6,/'PROVIDED STORAGE (IDIM)=',I6)
      IF (LSTM.GT.IDIM) STOP
C
C     ASSIGN TEMPORARY VARIABLE BMY EQUAL TO THE YIELD MOMENT (BMCAP)
C
      DO 20 MEMBN=1,NRM
20    BMY(MEMBN)=BMCAP(MEMBN)
C
C     ICOUNT IS THE NUMBER OF TIMES MAIN MSSM SUBROUTINE IS CALLED
C     ICOUNT IS INITIALIZED TO ZERO HERE.
C
      ICOUNT=0
C
C     SUBROUTINE MASS READS AND ASSIGNS MASSES TO NODES TO DETERMINE
C     THE MASS MATRIX.
C     SUBROUTINE MASS HAS INDEX NUMBERS STARTING AT 4001
C
      CALL MASS(NU,ND,AMASS,IUNIT,NRJ,NMASS,MDOF)
C
C    CALCULATE IF IDIM HAS BEEN SUFFICIENTLY DIMENSIONED
        IVAR1=(NU*NB)+NMASS
        IVAR2=NMASS*(NMODES+3)
        IF(IVAR1.GE.IDIM) WRITE(7,30)
        IF(IVAR2.GE.IDIM) WRITE(7,30)
30      FORMAT(' ','THE VALUE OF IDIM IS SMALLER THAN  RVPOW  REQUIRES')
C    REASSIGN OUTPUT TO TEMPORARY FILE 6
      IUNIT=6
C
C  IF ELASTIC ANALYSIS ONLY IS REQUIRED: RESET CONTROL FLAGS
C  SET FLAG TO INDICATE ONLY ONE ITERATION REQUIRED
C
      IF(INELAS.NE.0) GO TO 70
      WRITE(7,110)
      IUNIT=7
      IFLAG=1
      WRITE(7,110)
70    CONTINUE
C
C  SET THE MAXIMUM NUMBER OF ITERATIONS.
C
      IF(INELAS.NE.0) IMAX=INELAS
      IM=IMAX-1
C     I IS A PROGRAM LOCATION VARIABLE (SEE FLOWCHART)
C     IT SIGNIFIES NUMBER OF ITERATIONS PERFORMED.
      I=0
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C  BETA IS A NUMBER USED IN SPEEDING CONVERGENCE.  SHOULD BE A POSITIVE
C  NUMBER LESS THAN ONE.
C   A VALUE OF BETA OF ZERO EFFECTIVELY SHUTS OFF CONVERGENCE SPEEDING
C ROUTINE
      BETA=0.
C     SET ERROR RATIO OF MOMENTS OF YIELDED MEMBERS (BMERR).
C   A VALUE OF 0.05 HERE ENSURES YIELDED MEMBERS ARE WITHIN
C   5 PERCENT OF THEIR CAPACITY.
      BMERR=0.05
C
C  SET STOPPING VALUE FOR MINIMUM DAMAGE RATIO CHANGE BETWEEN SUCCESSIVE
C   ITERATIONS.  DAMERRE=0.01 ENSURES THAT THE MAXIMUM DAMAGE RATIO
C   CHANGE IN THE FINAL ITERATION IS ONE PERCENT FOR DAMAGE RATIOS
C    ABOVE 5.0
C  THOSE DAMAGE RATIOS BELOW 5.0 WILL HAVE A STOPPING CRITERION OF THEIR
C  ABSOLUTE VALUE DIFFERENCE BEING TEN TIMES THE RATIO.
C
       DAMERR=0.01
C  INITIALIZE ARRAY USED IN SPEEDING OF CONVERGENCE.
      DO 80 MEM=1,NRM
        DAMB(MEM)=DAMRAT(MEM)
80    CONTINUE
C
C-------------------------------------------------------
C     FINISHED INPUT OF DATA AND INITIAL ACTIVITIES.
C     BEGIN LOOP FOR MSS METHOD.
C-------------------------------------------------------
C
100   CONTINUE
C     INCREMENT ITERATION COUNTER.
      I=I+1
      WRITE(IUNIT,110)
110    FORMAT(' ',110('-'))
      WRITE(IUNIT,120) I
120   FORMAT('-','ITERATION NUMBER',I4)
C
C     SUBROUTINE BUILD COMPUTES THE MEMBER AND GLOBAL STIFFNESS MATRIX
C     SUBROUTINE BUILD IS INDEXED STARTING AT LINE 3001
C     WITH THE CALLING BELOW STIFFNESS MATRIX CANNOT HAVE GREATER THAN
C      1500 ENTRIES.
C
C CRMOM IS THE CRACKED MOMENT OF INERTIA OF THE SECTION.
      CALL BUILD(NU,NB,XM,YM,DM,NP,AREA,CRMOM,AV,E,G,DAMRAT,KL,KG,NRM,S,
     1      IDIM,EXTL,EXTG)
C
C  CALL SUBROUTINE TO CHECK ON THE CONDITIONING AND STABILITY OF
C  THE STIFFNESS MATRIX.
      CALL SCHECK(S,NU,NB,IDIM,IUNIT,SRATIO)
C
C     SUBROUTINE EIGEN COMPUTES THE FREQUENCIES AND MODES FOR THE
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C      SUBSTITUTE STRUCTURE.
C
       CALL EIGEN(NU,NB,S,IDIM,AMASS,EVAL,EVEC,NMODES,IUNIT,ISPEC,
      1     AMAX,ICOUNT,MDOF,INELAS)
C  INSERT HEADINGS FOR ITERATION PROGRESS OUTPUT AND TO
C  DIFFERENTIATE INELASTIC OUTPUT.
        IF(INELAS.EQ.O.OR.ICOUNT.NE.O) GO TO 105
        WRITE(7,110)
        WRITE(7,115)
 115    FORMAT(' ',// 25X,'INELASTIC RESULTS'//)
        WRITE(7,110)
        WRITE(7,90)
 90    FORMAT('-','ITERATION NO.',2X,'NO. ABOVE CAPACITY',2X,'DAMDIF',
      1        3X,'S MATRIX RATIO')
 105    CONTINUE
C     AFTER 10 ITERATIONS BETA IS REASSIGNED FROM 0.0 TO 0.25
       IF(I .GE. 9) BETA=0.80
C  ISIGN IS A COUNT OF THE NUMBER OF MEMBERS UNTOLERABLY ABOVE ULTIMATE.
C
C  FIND THE MEMBER WITH THE LARGEST DIFFERENCE IN DAMAGE RATIOS
C  BETWEEN THIS AND THE LAST ITERATION. USE VARIABLE 'DVARY'.
C  INITIALIZE DRFIFF TO ZERO HERE.
       DVARY=0.0
C
C  MOD3 IS THE MAIN SUBROUTINE FOR THE MSSM.  IT IS INDEXED FROM 6001.
C
       CALL MOD3(ICOUNT,ISPEC,NRJ,NRM,NU,NB,NMODES,S,500,ND,NP,XM,YM,DM,
      1     AREA,AV,CRMOM,DAMRAT,KL,KG,SDAMP,BMCAP,E,G,AMASS,EVEC,EVAL,
      2     AMAX,  ISIGN,IUNIT,BETA,BMERR,IFLAG,EXTL,EXTG,BETAM,DAMB,
      3     DVARY,INELAS,   DAMPIN,NPRINT)
C
C  IF ONLY DOING ELASTIC ANALYSIS THEN STOP PROGRAM
        IF(INELAS.EQ.O) GO TO 250
C
C      OUTPUT DAMAGE RATIOS ON UNIT 8
C     THESE ARE OUTPUT FOR EACH MEMBER AT EACH ITERATION.
C
C  OUTPUT NUMBER OF MEMBER IN EXCESS OF CAPACITY AND LARGEST
C  DIFFERENCE FROM PREVIOUS ITERATIONS DAMAGE RATIOS.
C  ALSO OUTPUT RATIO OF LARGEST TO SMALLEST MEMBER OF STIFFNES
C  MATRIX DIAGONAL (SRATIO)
        WRITE(7,130) I,ISIGN,DVARY,SRATIO
 130    FORMAT(' ',5X,I4,9X,I4,12X,F7.3,10X,E10.3)
        WRITE(8,140) (DAMRAT(MEMBRJ),MEMBRJ=1,NRM)
 140    FORMAT(' ',15F8.3)
C
C   IFLAG IS A FLAG USING INTEGER VALUES 1 AND O. MODIFIED
C   FROM O TO 1 WHEN NO MEMBERS ARE ABOVE CAPACITY.  IF ALL MEMBERS
C   ARE BELOW OR AT CAPACITY ONE FINAL ITERATION IS PERFORMED.
C    THE FOLLOWING LINES CHECK FOR YIELDING OF ALL MEMBERS AND THE
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C     MAXIMUM NUMBER OF ITERATIONS.
C
      IF(IFLAG.EQ.1 .AND. I.GE.IMAX) GO TO 180
      IF(IFLAG.EQ.1) GO TO 160
      IF(I.EQ.1 .AND. ISIGN.EQ.0) GO TO 200
      IF(I.GE.IM) GO TO 150
      ADERR=ABS(DVARY)
      IF(ISIGN.EQ.0.AND.ADERR.LT.DAMERR) GO TO 150
C
      GO TO 100
150   CONTINUE
C
C
      IFLAG=1
      IUNIT=7
      GO TO 100
160   CONTINUE
      WRITE(IUNIT,170) I
170   FORMAT('-',5X,'NO. OF ITERATIONS =',I5///)
      GO TO 220
180   CONTINUE
      WRITE(IUNIT,190) I
190   FORMAT('-',5X,'DOES NOT CONVERGE AFTER',I5,'   ITERATIONS'///)
      GO TO 220
200   CONTINUE
      ICOUNT=0
      IFLAG=1
      IUNIT=7
      WRITE(IUNIT,210)
210   FORMAT('-',5X,'MEMBERS DO NOT YIELD '///)
      GO TO 100
220   CONTINUE
      WRITE(IUNIT,230) BETA,BMERR
230   FORMAT('-',5X,'BETA=',F5.3,///5X,'BENDING MOMENT ERROR=',F8.6///)
      WRITE(IUNIT,240) DAMERR
240   FORMAT(' ','DAMAGE RATIO ERROR=', F6.3)
250   STOP
      END
C
C=================================================================
C
      SUBROUTINE CONTRL(TITLE,NRJ,NRM,E,G,IUNIT,AMAX,ISPEC,DAMPIN,
     1             INELAS,NMODES,NPRINT)
C
C=================================================================
C
      DIMENSION TITLE(20)
C
C  READ IN PROGRAM OPTIONS
C
```

```
      READ(5,10) INELAS,NMODES,NPRINT,ISPEC,AMAX,DAMPIN
10    FORMAT(4I5,2F10.5)
C  DAMPIN IS THE PROPORTION OF CRITICAL DAMPING USED IN ELASTIC
C  ANALYSIS OR THE FIRST ITERATION OF THE MSSM.
C
C  NPRINT IS A FLAG SET IF MODAL FORCES AND DISPLACEMENTS ARE REQUIRED
C  IF NPRINT=O ONLY RMS FORCES AND DISPLACEMENTS WILL BE PRINTED.
C  IF NPRINT IS GREATER THAN ZERO THAT NUMBER OF MODES (UP TO NMODES)
C  WILL HAVE THEIR FORCES AND DISPLACEMENTS PRINTED.
C
C  INELAS IS A FLAG INDICATING IF ONLY AN ELASTIC ANALYSIS IS REQUIRED
C  IF INELAS=O THEN ELASTIC ANALYSIS ONLY WILL BE PERFORMED.
C  IF INELAS IS GREATER THAN ZERO THEN THIS IS THE MAXIMUM NUMBER OF
C  ITERATIONS THAT WILL BE PERFORMED DURING INELASTIC ANALYSIS.
C
C  ECHO PRINT PROGRAM OPTIONS
      WRITE(IUNIT,20)
20    FORMAT(' ',//'*******PROGRAM OPTIONS*******'/)
      WRITE(IUNIT,30)NMODES
30    FORMAT(' ','MAXIMUM NUMBER OF MODES IN ANALYSIS', I4)
      IF(INELAS.EQ.O) WRITE(IUNIT,40)
40    FORMAT(' ','ELASTIC ANALYSIS REQUESTED')
      IF(INELAS.NE.O) WRITE(IUNIT,50) INELAS
50    FORMAT(' ','INELASTIC ANALYSIS MAXIMUM ITTERATIONS=',I4)
      IF(INELAS,EQ.O) WRITE(IUNIT,60) DAMPIN
60    FORMAT(' ','FRACTION OF CRITICAL DAMPING=',F6.4)
      IF(INELAS.GT.O) WRITE(IUNIT,70) DAMPIN
70    FORMAT(' ','INITIAL DAMPING RATIO= ',F6.3)
      WRITE(IUNIT,80) NPRINT
80    FORMAT(' ','NUMBER OF MODES TO HAVE OUTPUT PRINTED=',I3)
C
      WRITE(IUNIT,90)
      WRITE(IUNIT,100) AMAX
90    FORMAT('-','SEISMIC INPUT')
100   FORMAT('-', 'MAXIMUM ACCELERATION=',F5.3,' TIMES GRAVITY')
110   FORMAT(///110('-'))
      IF(ISPEC.EQ.1) WRITE (IUNIT,120)
      IF(ISPEC.EQ.2) WRITE (IUNIT,130)
      IF(ISPEC.EQ.3) WRITE (IUNIT,140)
      IF(ISPEC.EQ.4) WRITE(IUNIT,150)
      IF(ISPEC.GE.5) WRITE(IUNIT,160) ISPEC
      WRITE(IUNIT,110)
120   FORMAT(' ', 'SPECTRUM A USED')
130   FORMAT(' ', 'SPECTRUM B USED')
140   FORMAT(' ', 'SPECTRUM C USED')
150   FORMAT(' ','NATIONAL BUILDING CODE SPECTRUM USED')
160   FORMAT(' ', 'ERROR-SPECTRUM TYPE',I3,' IS NOT VALID')
      IF(ISPEC.NE.4) GO TO 200
      DPCNT=100.0*DAMPIN
C
```

```
      CALL SPECTR(ISPEC,DAMPIN,1.0,AMAX,SA,6.283,SABND,SVBND,SDBND)
C
      WRITE(IUNIT,170) DPCNT, SABND
  170 FORMAT(' ',F5.2,'% DAMPING SPECTRAL ACCEL. BOUND=',F6.3,' *G')
      WRITE(IUNIT,180) SDBND
  180 FORMAT(' ','                    DISPLACEMENT BOUND=',F6.3,' IN')
      WRITE(IUNIT,190) SVBND
  190 FORMAT(' ','                        VELOCITY BOUND=',F6.3,' IN/SEC')
C
C     READ IN TITLE
C
  200 READ (5,210)(TITLE(I),I=1,20)
C
C     READ IN NRJ,NRM,E,G
C
      READ (5,220) NRJ, NRM, E, G
      WRITE (IUNIT,230)(TITLE(I),I=1,20)
      WRITE (IUNIT,240) E, G
      WRITE (IUNIT,250)
      WRITE (IUNIT,260) NRJ, NRM
      WRITE(IUNIT,110)
C
C  CONVERT E AND G FROM KSI TO KSF.
       E=E*144.0
       G=G*144.0
C
      RETURN
  210 FORMAT(20A4)
  220 FORMAT(2I5,2F10.0)
  230 FORMAT('1',20A4)
  240 FORMAT('-',5X,'E =',F8.1,' KSI',5X,'G =',F8.1,' KSI')
  250 FORMAT(///110('*'))
  260 FORMAT('-','NO. OF JOINTS',' =',I5,10X,'NO. OF MEMBERS =',I5)
      END
C
C=================================================================
C
      SUBROUTINE SETUP(NRJ,NRM,E,G,XM,YM,DM,ND,NP,AREA,CRMOM,DAMRAT,AV,
     1            KL,KG,NU,NB,SDAMP,BMCAP,IUNIT,EXTL,EXTG)
C
C=================================================================
C
C
C      SET UP THE FRAME DATA
C
      DIMENSION KL(NRM), KG(NRM), AREA(NRM), CRMOM(NRM), SDAMP(NRM),
     1          DAMRAT(NRM), AV(NRM), ND(3,NRJ), NP(6,NRM), XM(NRM),
     2          YM(NRM),EXTL(NRM),EXTG(NRM), DM(NRM)
      DIMENSION X(100), Y(100), JNL(100), JNG(100), BMCAP(NRM)
C
```

```
C      E AND G IN KSF
C      X(I) AND Y(I) IN FEET
C   MEMBER EXTENSIONS EXTG AND EXTL ARE IN FEET.
C      AREA(I) IN SQ. INCHES:  CRMOM(I) IN INCHES**4
C      CONVERTED TO FOOT UNITS IN ROUTINE
       WRITE (IUNIT,230)
       WRITE (IUNIT,240)
C
C      READ IN JOINT DATA AND COMPUTE NO. OF DEGREES OF FREEDOM
C
       NU=1
C
       DO 50 I=1,NRJ
              READ (5,250) JN, ND(1,I), ND(2,I), ND(3,I), X(I), Y(I)
C
              DO 40 K=1,3
                  IF(ND(K,I)-1) 30,10,20
10                ND(K,I)=NU
                  NU=NU+1
                  GO TO 40
20                JNN=ND(K,I)
                  ND(K,I)=ND(K,JNN)
                  GO TO 40
30                CONTINUE
                  ND(K,I)=0
40            CONTINUE
C
C      PRINT JOINT DATA
C
              WRITE (IUNIT,260) I, X(I), Y(I), ND(1,I), ND(2,I), ND(3,I)
50     CONTINUE
C
       NU=NU-1
       WRITE (IUNIT,270)
       WRITE (IUNIT,280)
       WRITE (IUNIT,290)
C
C     READ IN MEMBER DATA AND COMPUTE THE HALF BANDWIDTH (NB)
C     HALF BANDWIDTH=MAX DEGREE OF FREEDOM-MIN DEGREE OF FREEDOM +1
C
C
       NB=0
C
       DO 190 MBR=1,NRM
              READ (5,300) MN,JNL(MBR),JNG(MBR),KL(MBR),KG(MBR),
     1  AREA(MBR),    CRMOM(MBR),AV(MBR),BMCAP(MBR),
     2  EXTL(MBR),EXTG(MBR)
C
C      IF DAMAGE RATIOS ARE LESS THAN ONE SET EQUAL TO ONE
C
```

```
                DAMRAT(MBR)=1.0
C  COMPUTE MEMBER LENGTH (DM)=LENGTH BETWEEN JOINTS-RIGID EXTENSIONS
                JL=JNL(MBR)
                JG=JNG(MBR)
                XM(MBR)=X(JG)-X(JL)
                YM(MBR)=Y(JG)-Y(JL)
                DM(MBR)=SQRT((XM(MBR))**2+(YM(MBR))**2)
          EXTSUM=EXTL(MBR)+EXTG(MBR)
          XM(MBR)=XM(MBR)*(1.0-EXTSUM/DM(MBR))
          YM(MBR)=YM(MBR)*(1.0-EXTSUM/DM(MBR))
C  RESET NEGATIVE VALUES OF ZERO TO ZERO
         IF(YM(MBR).GT.-0.01.AND.YM(MBR).LT.0.01) YM(MBR)=0.0
         IF(XM(MBR).GT.-0.01.AND.XM(MBR).LT.0.01) XM(MBR)=0.0
          DM(MBR)=DM(MBR)-EXTSUM
C
C  CHECK FOR NEGATIVE LENGTHS OF MEMBER
C  (PROBABLY CAUSED BY INCORRECT USE OF MEMBER EXTENSIONS)
C
         IF(DM(MBR).GT.0.0) GO TO 70
         WRITE(7,60) MBR
60       FORMAT(' ',///'PROGRAM HALTED:ZERO OR -VE LENGTH FOR MEMBER',I6)
         STOP
C
70       CONTINUE
C
             YLEN=YM(MBR)
C
C   PRINT ERROR MESSAGE IF ATTEMPT TO HAVE RIGID EXTENSIONS
C   ON VERTICAL MEMBERS.
          IF(EXTSUM.NE.0.0.AND.YLEN.GT.0.2) WRITE(7,80) I
80        FORMAT(' ','ERROR-HAVE END EXTENSIONS ON NON-HORIZONTAL
     1  MEMBER NO.',I3)
C  PRINT ERROR MESSAGE IF ATTEMPT TO HAVE RIGIND EXTENSIONS ON
C  A NON FIX-FIX TYPE MEMBER
          KLSUM=KL(MBR)+KG(MBR)
          IF(EXTSUM.NE.0.0.AND.KLSUM.NE.2) WRITE(7,90) MBR
90         FORMAT(' ','ERROR-HAVE RIGID EXTENSIONS ON HINGED MEMBER',I4)
C
C  GIVE MEMBERS INITIAL ELASTIC DAMPING
        SDAMP(MBR)=0.02
C
C  ASSIGN MEMBER DEGREES OF FREEDOM
                NP(1,MBR)=ND(1,JL)
                NP(2,MBR)=ND(2,JL)
                NP(3,MBR)=ND(3,JL)
                NP(4,MBR)=ND(1,JG)
                NP(5,MBR)=ND(2,JG)
                NP(6,MBR)=ND(3,JG)
C    DETERMINE THE HIGHEST DEGREE OF FREEDOM FOR EACH MEMBER STORING
C    THE RESULT IN 'MAX'
```

```
               MAX=0
C
               DO 120 K=1,6
                   IF(NP(K,MBR)-MAX) 110,110,100
100                MAX=NP(K,MBR)
110                CONTINUE
120            CONTINUE
C
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 PRESTENT POINT OF 1000.
C
               MIN=1000
C
               DO 160 K=1,6
                   IF(NP(K,MBR)) 150,150,130
130                IF(NP(K,MBR)-MIN) 140,150,150
140                MIN=NP(K,MBR)
150                CONTINUE
160            CONTINUE
C
               NBB=MAX-MIN+1
               IF(NBB-NB) 180,180,170
170            NB=NBB
180            CONTINUE
C
C      PRINT MEMBER DATA AND CONVERT TO FOOT UNITS.
C
               WRITE (IUNIT,310) MBR,JNL(MBR),JNG(MBR),EXTL(MBR),DM(MBR),
     1  EXTG(MBR),XM(MBR),YM(MBR),
     2  AREA(MBR),CRMOM(MBR),AV(MBR),BMCAP(MBR),KL(MBR),
     3  KG(MBR)
C
       AREA(MBR)=AREA(MBR)/144.0
       AV(MBR)=AV(MBR)/144.0
       CRMOM(MBR)=CRMOM(MBR)/20736.0
C
190    CONTINUE
C
C      PRINT THE NO. OF DEGREES OF FREEDOM AND THE HALF BANDWIDTH
C
       WRITE (IUNIT,320) NU
       WRITE (IUNIT,330) NB
C     OUTPUT THE ASSIGNED DEGREES OF FREEDOM.
        WRITE(IUNIT,200)
200    FORMAT(' ', '  MEMBER  NP1 NP2 NP3 NP4 NP5 NP6')
C
       DO 210 MEMBR=1,NRM
210    WRITE(IUNIT,220) MEMBR,(NP(IVAR,MEMBR),IVAR=1,6)
C
```

```
  220   FORMAT(' ',2X,I4,2X,6I4)
C
C
        RETURN
  230   FORMAT('-','JOINT DATA')
  240   FORMAT(/7X,'JN',3X,'X(FEET)',3X,'Y(FEET)',4X,'NDX',2X,'NDY',
     1         2X,'NDR')
  250   FORMAT(4I5,2F10.5)
  260   FORMAT(' ',5X,I4,2F10.3,2X,3I5)
  270   FORMAT('-','MEMBER DATA')
  280   FORMAT(/' MN JNL JNG   EXTL   LENGTH   EXTG  XM(FT)  YM(FT)',
     1         5X,'AREA   I(CRACKED)   AV',4X,'MOMENT',
     2         3X,'KL',1X,'KG')
  290   FORMAT(' ',19X,'(FEET)',29X,'(SQ.IN)',3X,'(IN**4)',
     1         3X,'(SQ.IN)',3X,'CAPACITY')
  300   FORMAT(5I5,F8.2,F12.3,2F10.3,2F6.3)
  310   FORMAT(' ',I3,2I4,F7.3,F9.4,F7.3,2F9.4,F8.1,F12.1,F8.3,F10.2,2I3)
  320   FORMAT('-','NO.OF DEGREES OF FREEDOM OF STRUCTURE =',I5)
  330   FORMAT(/'  HALF BANDWIDTH OF STIFFNESS MATRIX    =',I5)
        END
C
C===============================================================
C
        SUBROUTINE BUILD(NU,NB,XM,YM,DM,NP,AREA,CRMOM,AV,E,G,DAMRAT,
     1             KL,KG,NRM,S,IDIM,EXTL,EXTG)
C
C===============================================================
C
C
C  THIS SUBROUTINE WORKS IN DOUBLE PRECISION
C      THIS SUBROUTINE CALCULATES THE STIFFNESS MATRIX OF EACH
C      MEMBER AND ADDS IT INTO THE STRUCTURE STIFFNESS MATRIX.
C      THE FINAL STIFFNESS MATRIX S IS RETURNED.
C      THIS SUBROUTINE IS SIMILAR TO ONE THAT WOULD BE USED IN NORMAL
C      FRAME ANALYSIS.
C      DIFFERENCES INCLUDE USING CRACKED MOMENT OF INERTIA INSTEAD OF
C      THE GROSS SECTION.  DAMAGE RATIOS ARE USED AND FLEXTURAL
C      STIFFNESSES MODIFIED ACCORDING TO THESE RATIOS.
C      IDIM IS THE DIMENSIONING SIZE OF THE STRUCTURE STIFFNESS MATRIX.
C      INTERNAL FOOT UNITS FOR STIFFNESS MATRIX
C
        REAL*8 SM(21),S(IDIM)
        DIMENSION XM(NRM), YM(NRM), DM(NRM), NP(6,NRM), AREA(NRM),
     1            CRMOM(NRM), AV(NRM), DAMRAT(NRM), KL(NRM), KG(NRM)
        DIMENSION  EXTL(NRM), EXTG(NRM)
        REAL*8 RF,GMOD,CMOMI,DRATI,F,H
         REAL*8 LONE,LONEX,LONEY,LTWO,LTWOX,LTWOY,AVI
        REAL*8 YMI,DMI,DM2,XM2,YM2,XMI,AREAI,EMOD,XM2F,YM2F,XMYMF
        REAL*8 DBLE
C
```

```
C      ZERO STRUCTURE STIFFNESS MATRIX
C
       DO 10 I=1,IDIM
              S(I)=0.0D00
10     CONTINUE
C
C  REASSIGN YOUNGS MODULUS TO DOUBLE PRECISION VARIABLE EMOD
        EMOD=DBLE(E)
        GMOD=DBLE(G)
C
C      BEGIN MEMBER LOOP
C
       DO 200 I=1,NRM
C
C      ZERO MEMBER STIFFNESS NATRIX
C
              DO 20 J=1,21
                  SM(J)=0.0D00
20            CONTINUE
C
C  ASSIGN MEMBER PROPERTIES TO DOUBLE PRECESION VARIABLES
C
         LONE=DBLE(EXTL(I))
         LTWO=DBLE(EXTG(I))
         YMI=DBLE(YM(I))
         DMI=DBLE(DM(I))
          XMI=DBLE(XM(I))
          AREAI=DBLE(AREA(I))
          CMOMI=DBLE(CRMOM(I))
          DRATI=DBLE(DAMRAT(I))
          AVI=AV(I)
              DM2=DMI*DMI
              XM2=XMI*XMI
              YM2=YMI*YMI
              XMYM=XMI*YMI
              F=AREAI*EMOD/(DMI*DM2)
              H=0.0D00
C   SHEAR DEFLECTIONS ARE IGNORED WHENEVER G OR AV IS ZERO.
              IF(AV(I).EQ.0.0.OR.G.EQ.0.) GO TO 30
              H=12.0D00*EMOD*CMOMI/(AVI*GMOD*DM2)
30            XM2F=XM2*F
              YM2F=YM2*F
              XMYMF=XMYM*F
C
C      FILL IN PIN-PIN SECTION OF MEMBER STIFFNESS MATRIX
C
              SM(1)=XM2F
              SM(2)=XMYMF
              SM(4)=-XM2F
              SM(5)=-XMYMF
```

```
                 SM(7)=YM2F
                 SM(9)=-XMYMF
                 SM(10)=-YM2F
                 SM(16)=XM2F
                 SM(17)=XMYMF
                 SM(19)=YM2F
                 IF(KL(I)+KG(I)-1) 100,40,50
C
C    VALUES OF F CALCULATED HERE DIFFER FROM STANDARD BUILD SUBROUTINE
C    BY DEVIDING BY THE DAMAGE RATIOS.
C
40               F=3.0D00*EMOD*CMOMI/(DM2*DM2*DMI*(1.0D00+H/4.0D00))/DRATI
                 GO TO 60
50               F=12.0D00*EMOD*CMOMI/(DM2*DM2*DMI*(1.0D00+H))/DRATI
C  RF IS A FACTOR COMMON TO THE ENTIRE MATRIX FOR ADDITION OF STIFFNESS
C  DUE TO RIGID BEAM END EXTENSIONS.
                      RF=12.0D00*EMOD*CMOMI/(DM2*DM2)/DRATI
C
C     FILL IN TERMS WHICH ARE COMMON TO PIN-FIX,FIX-PIN,AND
C     FIX-FIX MEMBERS
C
60               XM2F=XM2*F
                 YM2F=YM2*F
                 XMYMF=XMYM*F
                 DM2F=DM2*F
             LONEY=LONE*YMI*RF
             LONEX=LONE*XMI*RF
             LTWOY=LTWO*YMI*RF
             LTWOX=LTWO*XMI*RF
C
                 SM(1)=SM(1)+YM2F
                 SM(2)=SM(2)-XMYMF
                 SM(4)=SM(4)-YM2F
                 SM(5)=SM(5)+XMYMF
                 SM(7)=SM(7)+XM2F
                 SM(9)=SM(9)+XMYMF
                 SM(10)=SM(10)-XM2F
                 SM(16)=SM(16)+YM2F
                 SM(17)=SM(17)-XMYMF
                 SM(19)=SM(19)+XM2F
                 IF(KL(I)-KG(I)) 70,80,90
C
C     FILL IN REMAINING PIN-FIX TERMS
C
70               SM(6)=-YMI*DM2F
                 SM(11)=XMI*DM2F
                 SM(18)=-SM(6)
                 SM(20)=-SM(11)
                 SM(21)=DM2*DM2F
                 GO TO 100
```

```
C
C     FILL IN REMAINING FIX-FIX TERMS
C
80          SM(3)=-YMI*DM2F*0.5D00
            SM(6)=SM(3)
            SM(8)=XMI*DM2F*0.5D00
            SM(11)=SM(8)
            SM(12)=DM2*DM2F*(4.0D00+H)/12.0D00
            SM(13)=-SM(3)
            SM(14)=-SM(8)
            SM(15)=DM2*DM2F*(2.0D00-H)/12.0D00
            SM(18)=-SM(3)
            SM(20)=-SM(8)
            SM(21)=SM(12)
C    ADD IN TERMS FOR RIGID END EXTENSIONS.
        SM(3)=SM(3)-(LONEY)
        SM(6)=SM(6)-(LTWOY)
        SM(8)=SM(8)+LONEX
        SM(11)=SM(11)+LTWOX
        SM(12)=SM(12)+(LONE*DMI*(DMI+LONE)*RF)
        SM(13)=SM(13)+LONEY
        SM(14)=SM(14)-LONEX
        SM(15)=SM(15)+((LONE*LTWO*DMI)+(DM2*(LONE+LTWO)/2.0D00))*RF
        SM(18)=SM(18)+LTWOY
        SM(20)=SM(20)-LTWOX
        SM(21)=SM(21)+(DM2*LTWO+(DMI*(LTWO*LTWO)))*RF
            GO TO 100
C
C     FILL IN REMAINING FIX-PIN TERMS
C
90          SM(3)=-YMI*DM2F
            SM(8)=XMI*DM2F
            SM(12)=DM2*DM2F
            SM(13)=-SM(3)
            SM(14)=-SM(8)
100         CONTINUE
C
C     ADD THE MEMBER STIFFNESS MATRIX SM INTO THE STRUCTURE
C     STIFFNESS MATRIX S.
C
            NB1=NB-1
C
            DO 190 J=1,6
                IF(NP(J,I)) 190,190,110
110             J1=(J-1)*(12-J)/2
C
                DO 180 L=J,6
                    IF(NP(L,I)) 180,180,120
120                 IF(NP(J,I)-NP(L,I)) 150,130,160
130                 IF(L-J) 140,150,140
```

```
 140              K=(NP(L,I)-1)*NB1+NP(J,I)
                  N=J1+L
                  S(K)=S(K)+2.0D00*SM(N)
                  GO TO 180
 150              K=(NP(J,I)-1)*NB1+NP(L,I)
                  GO TO 170
 160              K=(NP(L,I)-1)*NB1+NP(J,I)
 170              N=J1+L
                  S(K)=S(K)+SM(N)
 180           CONTINUE
C
 190        CONTINUE
C
 200  CONTINUE
C
      RETURN
      END
C
C==================================================================
C
      SUBROUTINE MASS(NU,ND,AMASS,IUNIT,NRJ,NMASS,MDOF)
C
C==================================================================
C
C
C     THIS SUBROUTINE SETS UP THE MASS MATRIX
C
C  ND(J,I)=DEGREES OF FREEDOM OF I TH JOINT
C  WTX,WTY,WTR=X-MASS,Y-MASS,ROT.MASS IN FORCE UNITS(KIPS OR IN-KIPS)
C  AMASS(I)=MASS MATRIX, I IS THE DEGREE OF FREEDOM OF APPLIED MASS
C     NMASS=NO.OF MASS POINTS
C
C     MASSES ARE LUMPED AT NODES.  THE MASS MATRIX IS DIAGONALIZED.
C
      DIMENSION ND(3,NRJ), MDOF(50), AMASS(NU)
C
C     READ IN NO. OF NODES WITH MASS
C
      READ (5,90) NMASS
      WRITE (IUNIT,100)
      WRITE (IUNIT,110) NMASS
      WRITE (IUNIT,120)
      WRITE (IUNIT,130)
C
C     ZERO MASS MATRIX
C
      DO 10 I=1,NU
              AMASS(I)=0.
 10   CONTINUE
C
```

```
C      READ IN X-MASS,Y-MASS AND ROT. MASS  (IN UNITS OF WEIGHT)
C
       DO 50 I=1,NMASS
              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.EQ.0) GO TO 20
              AMASS(N1)=AMASS(N1)+(WTX/32.2)
 20           IF(N2.EQ.0) GO TO 30
              AMASS(N2)=AMASS(N2)+(WTY/32.2)
 30           IF(N3.EQ.0) GO TO 40
              AMASS(N3)=AMASS(N3)+(WTR/32.2)
 40           CONTINUE
 50    CONTINUE
C
C     OUTPUT THE DEGREES OF FREEDOM WITH MASS AND ASSIGNED MASS.
C
       JCNT=1
       WRITE(IUNIT,70)
C
       DO 60 IDOF=1,NU
         RMASS=AMASS(IDOF)
         IF(RMASS.EQ.0.0) GO TO 60
         MDOF(JCNT)=IDOF
         WRITE(IUNIT,80) JCNT,MDOF(JCNT),RMASS
         JCNT=JCNT+1
 60    CONTINUE
C
C
 70    FORMAT('-','MASS NO. DOF',2X,'ASSIGNED MASS (KIP*SEC**2/FT)')
 80    FORMAT(' ',2X,I3,3X,I3,9X,F10.5)
       RETURN
 90    FORMAT(I5)
 100   FORMAT(///110('*'))
 110   FORMAT('-','NO. OF NODES WITH MASS',' =',I5)
 120   FORMAT(/7X,'JN',3X,'X-MASS',4X,'Y-MASS',2X,'ROT.MASS')
 130   FORMAT(' ',12X,'(KIPS)',4X,'(KIPS)',2X,'(IN-KIPS)')
 140   FORMAT(I5,3F10.0)
 150   FORMAT(' ',5X,I4,3F10.3)
       END
C
C=================================================================
C
       SUBROUTINE EIGEN(NU,NB,S,IDIM,AMASS,EVAL,EVEC,NMODES,IUNIT,
      1            ISPEC,AMAX,ICOUNT,MDOF,INELAS)
C
C=================================================================
C
```

```
C
C     THIS SUBROUTINE COMPUTES A SPECIFIED NO. OF NATURAL FREQUENCIES
C     AND ASSOCIATED MODE SHAPES
C
C     NU=NO. OF DEGREES OF FREEDOM
C     NB=HALF BANDWIDTH
C     NMODES=NO. OF MODE SHAPES TO BE COMPUTED
C     IF NMODES IS ZERO OR IS GREATER THAN THE NUMBER OF STRUCTURE
C      MASSES THEN NMODES WILL BE ASSIGNED THE NUMBER OF STRUCTURE
C MASSES.
C     AMASS(I)=MASS MATRIX   MCOUNT=NUMBER OF NONZERO MASSES
C     S(I)=STIFFNESS MATRIX STORED BY COLUMNS
C     EVAL(I)=NATURAL FREQUENCIES
C     EVEC(I,J)=MODE SHAPES
C
       REAL*8 DVEC(300,10),DVAL(10),CMASS(300),SD(2000)
       REAL*8 S(IDIM)
      DIMENSION  AMASS(NU), EVAL(NMODES), EVEC(50,NMODES),
     1          MDOF(50)
       REAL*8 DBLE
C
C     ZERO DUMMY MASS MATRIX CMASS
       DO 10 ITRY=1,100
10     CMASS(ITRY)=0.0
C
C  DEBUG ON-OFF SWITCH FOLLOWS.
       IOFF=0
       ION=1
       IDEBUG=ION
C
C     COMPUTE THE NUMBER OF NONZERO MASS MATRIX ENTRIES
C
      MCOUNT=0
C
      DO 20 I=1,NU
             CMASS(I)=DBLE(AMASS(I))
             IF(AMASS(I).EQ.0.) GO TO 20
             MCOUNT=MCOUNT+1
20    CONTINUE
C
      IF(NMODES.GT.MCOUNT) NMODES=MCOUNT
      IF(NMODES.EQ.0) NMODES=MCOUNT
       IF(IUNIT.EQ.6.AND.ICOUNT.GT.25) GO TO 30
      WRITE (IUNIT,160) NMODES
30      CONTINUE
C
C     CALL PRITZ TO COMPUTE EIGENVALUES AND EIGENVECTORS
C    CREATE A DUPLICATE STRUCTURE MATRIX (SD) (DESTROYED IN PRITZ)
C
C CALCULATE USEFUL LENGTH OF STIFFNESS MATRIX (LSTM)
```

```
      LSTM=(NU)*NB
C
     DO 40 I=1,LSTM
       SD(I)=S(I)
40    CONTINUE
C   SET CONVERGENCE CRITERIA FOR PRITZ. MAKE NEGATIVE IF RESIDUALS NOT
C DESIRED.
C
     DEPS=1.0D-10
     IF(IUNIT.NE.7) DEPS=(-1.0D00)*DEPS
C
C
C  CALL EIGENVALUE FINDING ROUTINE
     CALL PRITZ(SD,CMASS,NU,NB,1,DVAL,DVEC,300,NMODES,DEPS,&140)
C
C  CONVERT MATRICES TO SINGLE PRECESION
C
C     PRINT EIGENVALUES AND EIGENVECTORS(MODE SHAPES)
C     EIGENVALUES (EVAL) ARE THE VALUES OF OMEGA SQUARED.
C
C     SKIP PRINTING INTERMEDIATE DATA AFTER SEVERAL CYCLES.
      IF(ICOUNT.GT.3.AND.IUNIT.EQ.6) GO TO 70
      WRITE (IUNIT,170)
      WRITE (IUNIT,210) NMODES
      WRITE (IUNIT,230)(I,I=1,NMODES)
C
      DO 60 ID=1,NU
        WRITE(IUNIT,50) ID,(DVEC(ID,J), J=1,NMODES)
50      FORMAT(' ',I3,10F11.6)
60    CONTINUE
C
70    CONTINUE
C     ALSO CONVERT MEMBERS OF EVAL FROM OMEGA SQUARED TO OMEGA
C
C  CONVERT EIGENVECTORS TO ONLY INCLUDE DEGREES OF FREDOM WITH MASS
C  ASSIGNED TO THEM
      DO 90 MAS=1,MCOUNT
        IVAR=MDOF(MAS)
C
        DO 80 MOD=1,NMODES
          EVEC(MAS,MOD)=SNGL(DVEC(IVAR,MOD))
80      CONTINUE
C
90    CONTINUE
C
      IF(ICOUNT.EQ.0) WRITE(7,900)
900   FORMAT(' ',// '------INITIAL ELASTIC PERIOD------')
      IF(ICOUNT.EQ.0) IUNIT=7
      WRITE (IUNIT,180)
      WRITE (IUNIT,190)
```

```
C
C     COMPUTE FREQUENCIES AND PERIODS
      DO 100 JUICE=1,NMODES
  100 EVAL(JUICE)=SNGL(DVAL(JUICE))
C
      DO 110 I=1,NMODES
             EVAL1=EVAL(I)
             EVAL(I)=SQRT(EVAL1)
        WN=EVAL(I)
        PERIOD=6.283153/WN
        FREQ=1/PERIOD
        IF(ICOUNT.GT.25.AND.IUNIT.EQ.6) GO TO 110
         CALL SPECTR(ISPEC,0.02,PERIOD,AMAX,SA,WN,SABND,SVBND,SDBND)
             WRITE (IUNIT,200) I, EVAL1, EVAL(I), FREQ, PERIOD, SA
  110 CONTINUE
      IF(ICOUNT.EQ.0.AND.INELAS.NE.0) IUNIT=6
C
      IF(ICOUNT.GT.5.AND.IUNIT.EQ.6) GO TO 130
      WRITE (IUNIT,220) NMODES
      WRITE (IUNIT,240)(I,I=1,NMODES)
C
      DO 120 I=1,MCOUNT
             WRITE (IUNIT,50) I,(EVEC(I,J),J=1,NMODES)
  120 CONTINUE
C
  130   CONTINUE
C
      RETURN
  140  WRITE(IUNIT,150)
  150  FORMAT(' ','CRAPOUT IN PRITZ')
  160 FORMAT('-','NO. OF MODES TO BE ANALIZED =',I5///110('*')////)
  170 FORMAT(///110('*'))
  180 FORMAT(/5X,'MODES',4X,'EIGENVALUES',6X,'NATURAL FREQUENCIES',
     1        13X,'PERIODS',10X,'SA')
  190 FORMAT(' ',30X,'(RAD/SEC)',5X,'(CYCS/SEC)',8X,'(SECS)',
     1        4X,'(2 PERCENT DAMPING)')
  200   FORMAT(' ',5X,I5,5F15.4)
  210   FORMAT(/'TOTAL MODE SHAPES CORRESPONDING TO FIRST',I5,
     1        1X,'FREQUENCIES')
  220  FORMAT(/'MASS MODE SHAPES CORRESPONDING TO FIRST',I5,1X,
     1        'FREQUENCIES')
  230   FORMAT(/' DOF',I8,9I11)
  240   FORMAT(/'MASS',10I11)
  250 FORMAT(' ',10F12.6)
       RETURN
      END
C
C================================================================
C
      SUBROUTINE MOD3(ICOUNT,ISPEC,NRJ,NRM,NU,NB,NMODES,S,IDIM,ND,NP,XM,
```

```
     1             YM,DM,AREA,AV,CRMOM,DAMRAT,KL,KG,SDAMP,BMCAP,E,G,AMASS,
     2             EVEC,EVAL, AMAX,ISIGN,IUNIT,BETA,BMERR,IFLAG,EXTL,EXTG,
     3             BETAM,DAMB,DVARY,      INELAS,DAMPIN,NPRINT)
C
C=================================================================
C
C
C     SUBSTITUTE STRUCTURE METHOD FOR RETROFIT
C     THIS SUBROUTINE COMPUTES JOINT DISPLACEMENTS AND MEMBER FORCES
C     NEW DAMAGE RATIOS WILL BE CALCULATED AND RETURNED.
      REAL*8 S(IDIM),DF(100)
C
      DIMENSION ND(3,NRJ), NP(6,NRM), XM(NRM), YM(NRM), DM(NRM),
     1          AREA(NRM), CRMOM(NRM), DAMRAT(NRM), KL(NRM), KG(NRM),
     2          AMASS(NU),SUMDAM(100),EVEC(50,NMODES), EVAL(NMODES),
     3          SDAMP(NRM), AV(NRM), ZETA(10), PI(100)
      DIMENSION BMASS(50), IDOF(50), ALPHA(20), RMS(7,100),
     1          F(300),EXTL(NRM),EXTG(NRM), D(6)
      DIMENSION BMCAP(NRM),DAMB(NRM), BETAM(NMODES)
      REAL*8 DRATIO,DET
C     CALCULATE THE MODAL PARTICIPATION FACTOR
C     JJ=TEMPORARY VARIBLE USED IN NEXT LOOP ONLY.
C
 10   FORMAT(' ', 'ICOUNT=', I3)
 20   CONTINUE
      JJ=1
C
      DO 30 JDOF=1,NU
            IF(AMASS(JDOF).EQ.0.) GO TO 30
            BMASS(JJ)=AMASS(JDOF)
            IDOF(JJ)=JDOF
            JJ=JJ+1
 30   CONTINUE
C
      MCOUNT=JJ-1
C
      DO 70 MODEY=1,NMODES
            AMT=0.
            AMB=0.
C
C     EIGEN VALUES ARE STORED AS FOLLOWS EVEC(MASS NO.,MODE NO.)
C
            DO 60 JAM=1,MCOUNT
               AMT=AMT+BMASS(JAM)*EVEC(JAM,MODEY)
               AMB=AMB+BMASS(JAM)*((EVEC(JAM,MODEY))**2)
 60         CONTINUE
C
            ALPHA(MODEY)=AMT/AMB
 70   CONTINUE
C
```

```
           IF(ICOUNT.GT.25.AND.IUNIT.EQ.6) GO TO 90
           WRITE (IUNIT,810)
C
           DO 80 MODEX=1,NMODES
                  WRITE (IUNIT,820) MODEX, ALPHA(MODEX)
80         CONTINUE
C
90         CONTINUE
C
C          WHEN KK=1, MODAL FORCES FOR UNDAMPED SUBSTITUTE STRUCTURE ARE
C          COMPUTED.  THEY ARE USED TO COMPUTE 'SMEARED' DAMPING VALUES,
C          WHICH ARE USED TO CALCULATE THE ACTUAL RESPONSE OF THE SUBSTITUTE
C          STRUCTURE
C
           INDEX=1
C
           DO 800 KK=1,2
C
C   SET PRINT FLAG FOR MODAL OUTPUT (0=OFF)
              INTPR=1
             IF(KK.EQ.1) INTPR=0
               IF(IFLAG.EQ.0.OR.NPRINT.EQ.0) INTPR=0
             IF(ICOUNT.EQ.0) GO TO 780
                  SHRMS=0.
C
C          ZERO  RMS(J,I)
C
                  DO 110 I=1,100
C
                      DO 100 J=1,7
                          RMS(J,I)=0.
100                   CONTINUE
C
110               CONTINUE
C
C          OUTPUT THE SMEARED DAMPING RATIOS (FOR DAMPED CASES)
               IF(IUNIT.EQ.6.AND.ICOUNT.GT.25) GO TO 130
               IF(KK.LT.2) GO TO 130
C
             WRITE(IUNIT,140)
C
             DO 120 MODEC=1,NMODES
                  WRITE(IUNIT,150) MODEC,BETAM(MODEC)
120          CONTINUE
C
130             CONTINUE
140          FORMAT('-','MODE',2X,'SMEARED DAMPING RATIO')
150          FORMAT(' ',1X,I3,7X,F10.5)
C
C           CALCULATE THE MODAL DISPLACEMENT VECTOR
```

```
C     FIRST ZERO TEMPORARY VARIABLE ZETA USED IN CALCULATING DAMPING.
         DO 160 MODEJ=1,NMODES
           ZETA(MODEJ)=0.0
160      CONTINUE
C
              DO 570 MODEN=1,NMODES
C  LIST MEMBER FORCES IF DOING ELASTIC ANLYSIS ONLY
C
           IF(INTPR.EQ.0) GO TO 180
          IF(NPRINT.LT.MODEN) GO TO 180
           WRITE(IUNIT,840)
           WRITE(IUNIT,170) MODEN
170        FORMAT(' ','MODE NUMBER',I3,' MODAL FORCES AND DISPLACEMENTS
     1')
           WRITE(IUNIT,830)
180        CONTINUE
C
C  CHECK IF MODAL PARTICIPATION FACTOR IS ZERO
C  IF ALPHA IS ZERO MODAL FORCES AND DISPLACEMENTS WILL ALSO BE ZERO
C
           IF(ALPHA(MODEN).NE.0.0) GO TO 200
           WRITE(IUNIT,190)
190        FORMAT(/ ' MODAL PARTICIPATION ,FORCES AND DISPL.=ZERO')
           GO TO 570
200        CONTINUE
C
C     CALCULATE NATURAL PERIOD AND CALL SPECTA
C
           TN=6.28318531/(EVAL(MODEN))
           WN=EVAL(MODEN)
                 DAMP=BETAM(MODEN)
                 CALL SPECTR(ISPEC,DAMP,TN,AMAX,SA,WN,SABND,SVBND,SDBND)
C
C     ZERO LOAD VECTOR
C
                 DO 210 J=1,NU
                    F(J)=0.
210              CONTINUE
C
                 FF=0.
C
C     COMPUTE LOAD VECTOR
C
                 FAC=SA*ALPHA(MODEN)*32.2
C
C     NOTE THAT AS THESE FORCES ARE BEING GENERATED FROM A
C     LATERAL EXCITATION SPECTRUM THAT ONLY 'X MASSES' SHOULD
C     BE USED.  IN OTHER WORDS LATERAL ACCELERATION SHOULD NOT
C     CAUSE NON HORIZONTAL INERTIA FORCES DIRECTLY.
C
```

```
                    DO 220 J=1,MCOUNT
                        I1=IDOF(J)
                        F(I1)=EVEC(J,MODEN)*FAC*AMASS(I1)
                        FF=FF+F(I1)
220                 CONTINUE
C
C     CALCULATE THE BASE SHEAR
C
                    IF(KK.NE.2) GO TO 230
                    SHRMS=SHRMS+FF**2
                    IF(MODEN.LT.NMODES) GO TO 230
                    SHRMS=SQRT(SHRMS)
230                 CONTINUE
C  CONVERT SINGLE PRECISION FORCE MATRIX TO DOUBLE PRECISION
            DO 240 IFREE=1,100
              DF(IFREE)=DBLE(F(IFREE))
240         CONTINUE
C
C     COMPUTE DEFLECTIONS BY CALLING SUBROUTINE DFBAND
            LSTM=NU*NB
C  NOTE THAT NO SOLUTION IMPROVING ITERATIONS WILL BE PERFORMED.
C  SCALING WILL BE PERFORMED TO IMPROVE THE SOLUTION WHEN NSCALE.NE.O
C
            NSCALE=1
C
                    DRATIO=1.OD-16
                    CALL DFBAND(S,DF,NU,NB,INDEX,DRATIO,DET,JEXP,NSCALE)
C    DFBAND EXITS WITH F BEING THE DISPLACEMENT MATRIX
C
C  CONVERT DOUBLE PRECISION DISPLACEMENTS TO SINGLE PRECISION
            DO 250 JFREE=1,100
              F(JFREE)=SNGL(DF(JFREE))
250         CONTINUE
C
                    INDEX=INDEX+1
C
C
C    CALCULATE RMS DISPLACEMENTS.
                    DO 290 JNT=1,NRJ
                        DX=O.
                        DY=O.
                        DR=O.
                        N1=ND(1,JNT)
                        N2=ND(2,JNT)
                        N3=ND(3,JNT)
                        IF(N1.EQ.O) GO TO 260
                        DX=F(N1)
                        RMS(1,JNT)=RMS(1,JNT)+DX**2
260                     CONTINUE
                        IF(N2.EQ.O) GO TO 270
```

```
                     DY=F(N2)
                     RMS(2,JNT)=RMS(2,JNT)+DY**2
 270                 CONTINUE
                     IF(N3.EQ.0) GO TO 280
                     DR=F(N3)
                     RMS(3,JNT)=RMS(3,JNT)+DR**2
 280                 CONTINUE
              IF(INTPR.EQ.0) GO TO 290
             IF(NPRINT.LT.MODEN) GO TO 290
C  OUTPUT MODAL DEFLECTIONS FOR REQUIRED MODES
              IF(N1.EQ.0) DX=0.0
              IF(N2.EQ.0) DY=0.0
              IF(N3.EQ.0) DR=0.0
              WRITE(IUNIT,860) JNT,DX,DY,DR
C
 290               CONTINUE
C---------------------------------------------------------------
C  AT THIS STAGE RMS(1,JNT)=(RMS DISPLACEMENT)SQUARED OF X DISPLACEMENT.
C  COMPUTE MEMBER FORCES USING DISPLACEMENTS FROM INDIVIDUAL MODES
C  NOTE THAT 'ENGINEERING' SIGN CONVENTION IS USED HERE.
C---------------------------------------------------------------
                   SIGPI=0.
C  INSERT MODAL MEMBER FORCE HEADINGS BEFORE STARTING MEMBER FORCE LOOP.
C
             IF(INTPR.NE.0.AND.NPRINT.GE.MODEN) WRITE(IUNIT,300)
 300        FORMAT(' ',/8X,'MN',10X,'AXIAL',10X,'SHEAR',11X,'BML',12X,
     1             'BMG',        /21X,'KIPS',12X,'KIPS',2(9X,'(K-FT)'))
C
C----------------------------
C
                   DO 460 I=1,NRM
C
C----------------------------
C
C     XL AND YL =X AND Y COMPONENTS OF MEMBER LENGTH RESPECTIVELY
C     DL IS TRUE LENGTH OF MEMBER
C    BMG IS THE BENDING MOMENT AT GREATER JOINT NO. END OF MEMBER.
C     BML IS THE BENDING MOMENT AT THE LESSER JOINT NO. END.
C
                     XL=XM(I)
                     YL=YM(I)
                     DL=DM(I)
                     AVI=AV(I)
C
                     DO 340 MEMDOF=1,6
                        N1=NP(MEMDOF,I)
                        IF(N1) 320,320,310
 310                    D(MEMDOF)=F(N1)
                        GO TO 330
 320                    D(MEMDOF)=0.
```

```
330                          CONTINUE
340                      CONTINUE
C
C    MODIFY END DISPLACEMENTS FOR HORIZONTAL MEMBERS WITH END EXTENSIONS
C    FORMULA ONLY WORKS FOR HORIZONTAL MEMBERS
              N3=NP(3,I)
             IF(N3.EQ.0) GO TO 350
              D(2)=D(2)+(F(N3))*EXTL(I)
350            CONTINUE
              N6=NP(6,I)
              IF(N6.EQ.0) GO TO 360
              D(5)=D(5)-(F(N6))*EXTG(I)
360           CONTINUE
C   PRINT OUT MEMBER END DISPLACEMENTS FOR DEBUG.
                IF(ICOUNT.GT.1) GO TO 380
              WRITE(6,370) I,(D(M),M=1,6)
370           FORMAT(' ', 'MEMB NO.=',I3,'DISPL=',6F10.5)
380            CONTINUE
                AXIAL=(AREA(I)*E/DL**2)*(D(4)*XL+D(5)*YL-D(1)*XL-D(2)*YL)
C   EISI=ASSUMED STIFFNESS IN SUBSTITUTE FRAME ELEMENT I
                 EISI=CRMOM(I)*E/DAMRAT(I)
C
C   GFACT=FACTOR TO COMPUTE EFFECT OF SHEAR DEFL. ON MEMBER FORCES
C     GFACT=0.0 IMPLIES THAT NO SHEAR DEFLECTION INCLUDED.
              GFACT=0.0
              IF(AVI.EQ.0.0.OR.G.EQ.0.0) GO TO 390
              GFACT=12.0*EISI/(AVI*G*DL*DL)
390           CONTINUE
C
C  ASSIGN DISPLACEMENTS TO THEIR RESPECTIVE MEMBER DEGREES OF FREEDOM
C    CHECK FOR PIN-PIN MEMBERS
                     IF(KL(I).EQ.0 .AND. KG(I).EQ.0) GO TO 420
             DELT=((D(5)-D(2))*XL+(D(1)-D(4))*YL)/DL
             BML=(2.0*EISI/(DL*(1.0+GFACT)))*((3.0*DELT/DL)
     1       -(D(6)*(1.0-GFACT/2.0))-(2.0*D(3)*(1.0+GFACT/4.0)))
             SHEAR=(6.0*EISI/(DL*DL))*((D(3)+D(6)-(2.0*DELT/DL))/(1.0+
     1       GFACT))
                     BMG=BML+SHEAR*DL
                     IF(KL(I)-KG(I)) 400,430,410
C    ADJUST PIN-FIX MEMBER FORCES.
400                  BMG=BMG+BML*(1.0-GFACT/2.0)/(2.0*(1.0+GFACT/4.0))
                     SHEAR=SHEAR+1.5*BML/(DL)
                     BML=0.
                     GO TO 430
C    ADJUST FIX-PIN MEMBER FORCES.
410                  BML=BML+BMG*(1.0-GFACT/2.0)/(2.0*(1.0+GFACT/4.0))
                     SHEAR=SHEAR-1.5*BMG/(DL)
                     BMG=0.
                     GO TO 430
C    FILL IN MEMBER FORCES FOR PIN-PIN MEMBERS.
```

```
420                     BMG=0.
                        BML=0.
                        SHEAR=0.
430                     CONTINUE
C
C       COMPUTE THE RELATIVE FLEXURAL STRAIN ENERGY
C
                        IF(KK.NE.1) GO TO 440
                        PI(I)=(BML**2+BMG**2+BML*BMG)*DL/(6.*EISI)
                        SIGPI=SIGPI+PI(I)
440                     CONTINUE
C
C  PRINT OUT FORCES FOR EACH MEMBER IF ELASTIC CASE DESIRED.
                IF(INTPR.EQ.0) GO TO 450
               IF(NPRINT.GE.MODEN) WRITE(IUNIT,900) I,AXIAL,SHEAR,BML,BMG
450             CONTINUE
C
C       ACCUMULATE ABSOLUTE SUM AND RMS SUM
C
                        RMS(4,I)=RMS(4,I)+AXIAL**2
                        RMS(5,I)=RMS(5,I)+SHEAR**2
                        RMS(6,I)=RMS(6,I)+BML**2
                        RMS(7,I)=RMS(7,I)+BMG**2
460                 CONTINUE
C
C       COMPUTE THE SMEARED DAMPING FOR EACH MODE
C
                    IF(KK.NE.1) GO TO 540
C
C   SUMDAM= THE PRODUCT OF MEMBER STRAIN ENERGY*MEMBER DAMPING.
                    DO 470 I=1,NRM
                SUMDAM(I)=PI(I)*SDAMP(I)
                        ZETA(MODEN)=ZETA(MODEN)+SUMDAM(I)
470                 CONTINUE
C
C     BETAM=SMEARED SUBSTITUTE DAMPING FOR THE M TH MODE.
                    BETAM(MODEN)=ZETA(MODEN)/SIGPI
C
C  PRINT DAMPING INFORMATION FROM FINAL ITERATION.
C
            IF(IFLAG.NE.1) GO TO 520
            WRITE(6,480)SIGPI,MODEN,BETAM(MODEN)
480          FORMAT(' ','TOTAL FLEX. STR. ENERGY=',F10.3,3X,'MODE NUMBER',
     1            I2,3X,'SMEARED DAMPING FACTOR=',F7.5)
             WRITE(6,490)
C
             DO 510 MEMB=1,NRM
490            FORMAT(' ','MEMBER NO.',3X,'STRAIN ENERGY',3X,
     1               'MEMBER DAMPING',  3X,'MEMBER DAMPING*STRAIN ENERGY')
               WRITE(6,500) MEMB,PI(MEMB),SDAMP(MEMB),SUMDAM(MEMB)
```

```
 500          FORMAT(' ',4X,I2,10X,E10.3,8X,E10.3,13X,F11.7)
 510        CONTINUE
 C
 520         CONTINUE
 C
           IF(SIGPI.EQ.0.0) WRITE(IUNIT,530)
 530       FORMAT(' ','ERROR-ZERO DEVIDE WHILE CALCULATING SMEARED DAMPIN
      1G')
 540                CONTINUE
 C
 C  COMPUTE AND WRITE MODAL CONTRIBUTION FACTOR
              CONMOD=SA*ALPHA(MODEN)
            WRITE(IUNIT,550) MODEN, CONMOD
 550        FORMAT(' ','MODE ',I3,3X,'CONTRIBUTION FACTOR=',F8.5)
 C  OUTPUT SPECTRAL ACCELERATION.
 C
            IF(INTPR.EQ.0.OR.MODEN.GT.NPRINT) GO TO 570
            WRITE(IUNIT,560) DAMP, TN,SA
 560       FORMAT(' ','DAMPING=',F6.4,' PERIOD=',F6.4,' SEC. SA=',F5.3)
 570          CONTINUE
 C
          IF(KK.EQ.1.AND.ICOUNT.LT.2) GO TO 580
              IF(KK.EQ.1) GO TO 800
 C
 580      CONTINUE
 C
 C      PRINT RMS DISPLACEMENTS AND FORCES
 C
          IF(IUNIT.EQ.6.AND.ICOUNT.GT.25) GO TO 590
              WRITE (IUNIT,840)
 C      OUTPUT THE COUNT OF ENTRANCES INTO MOD3
        WRITE(6,10) ICOUNT
              WRITE (IUNIT,850)
              WRITE (IUNIT,830)
 590        CONTINUE
 C
 C      CONVERT SQUARE OF RMS DISPLACEMENTS TO RMS DISPLACEMENTS.
              DO 610 I=1,NRJ
 C
                  DO 600 J=1,3
                      SCRAT=RMS(J,I)
                      RMS(J,I)=SQRT(SCRAT)
 600              CONTINUE
 C
            IF(ICOUNT.GT.25.AND.IUNIT.EQ.6) GO TO 610
 C
                  WRITE (IUNIT,860) I, (RMS(J,I),J=1,3)
 610          CONTINUE
 C
 C      MODIFY DAMAGE RATIOS
```

```
C
        IF(ICOUNT.GT.25.AND.IUNIT.EQ.6) GO TO 630
            WRITE (IUNIT,870)
            WRITE (IUNIT,880) SHRMS
620         CONTINUE
       IF(ICOUNT.GT.25.AND.IUNIT.EQ.6) GO TO 630
            WRITE (IUNIT,890)
630    CONTINUE
C
C ISIGN IS A COUNT OF THE NUMBER OF MEMBERS WITH WHICH THE RATIO OF
C THE ABSOLUTE VALUE OF THE DIFFERENCE BETWEEN THE LARGEST RMS
C BENDING MOMENT AND ULTIMATE MOMENT TO ULTIMATE MOMENT IS IN
C EXCESS OF 'BMERR'.
C ISIGN IS INITIALIZED TO ZERO HERE.
C
            ISIGN=0
C
            DO 770 MEM=1,NRM
C    FIND THE BIGGEST OF THE SQUARE OF THE RMS BENDING MOMENT(=BIG)
                IF(RMS(6,MEM)-RMS(7,MEM))640,640,650
640             BIG=RMS(7,MEM)
                GO TO 660
650             BIG=RMS(6,MEM)
660             CONTINUE
         IF(KK.EQ.1)GO TO 750
C    TAKE SQUARE ROOT TO GIVE RMS BENDING MOMENT.
                BMBIG=SQRT(BIG)
C
C  SET DAMOLD AS THE DAMAGE RATIO IN THE (I-2)TH ITERATION
C      DAMB AS THE DAMAGE RATIO IN THE (I-1)TH ITERATION.
C
                DAMOLD=DAMB(MEM)
         DAMB(MEM)=DAMRAT(MEM)
C  CALCULATE NEW DAMAGE RATIO
C
                DAMRAT(MEM)=BMBIG/BMCAP(MEM)*DAMRAT(MEM)
C  DO NOT ALTER DAMAGE RATIOS OF LESS THAN UNITY, AS THEY ARE RESET AT
C  END OF ROUTINE.
         IF(DAMRAT(MEM).LT.1.0) GO TO 730
C
C
C  CONVERGENCE SPEEDING ROUTINE FOLLOWS.
         IF(DAMRAT(MEM).LT.5.0) DERROR=(DAMRAT(MEM)-DAMB(MEM))/10.0
         IF(DAMRAT(MEM).GE.5.0) DERROR=(DAMRAT(MEM)-DAMB(MEM))/DAMRAT(
     1   MEM)
         ADIFF=ABS(DERROR)
         IF(ADIFF.GT.DVARY) DVARY=DERROR
C
          DAMDIF=DAMRAT(MEM)-DAMB(MEM)
C
```

```
            IF(DAMOLD-DAMB(MEM)) 670,730,700
  670       CONTINUE
            IF(DAMDIF) 690,730,680
  680       DAMRAT(MEM)=DAMRAT(MEM)+BETA*(DAMDIF)
            GO TO 730
  690       DAMRAT(MEM)=DAMRAT(MEM)-BETA*(DAMDIF)
            GO TO 730
  700       CONTINUE
            IF(DAMDIF) 720,730,710
  710       CONTINUE
            DAMRAT(MEM)=DAMRAT(MEM)-BETA*(DAMDIF)
            GO TO 730
  720       CONTINUE
            DAMRAT(MEM)=DAMRAT(MEM)+BETA*(DAMDIF)
  730       CONTINUE
            IF(DAMRAT(MEM).LT.1.0.AND.IFLAG.NE.1) DAMRAT(MEM)=1.0
C
C  DAMAGE RATIOS CANNOT BE LESS THAN 1.0
C  IN LAST ITERATION SKIP RESETTING DAMAGE RATIOS LESS THAN UNITY
C
                     IF(DAMRAT(MEM).LE.1.0) GO TO 740
                     CHECK=ABS(BMBIG-BMCAP(MEM))/BMCAP(MEM)
                     IF(CHECK.GT.BMERR) ISIGN=ISIGN+1
  740                CONTINUE
C     COMPUTE DAMPING VALUE FOR THE MEMBER
                     SDAMP(MEM)=0.02+0.2*(1.-1./SQRT(DAMRAT(MEM)))
C
  750          CONTINUE
C
C     CONVERT SQUARE OF RMS AXIAL, SHEAR AND MOMENT TO RMS VALUE.
                     DO 760 J=4,7
                        RMS(J,MEM)=SQRT(RMS(J,MEM))
  760                CONTINUE
C
C     OUTPUT THE RMS AXIAL SHEAR AND MOMENT.
                   IF(ICOUNT.GT.25.AND.IUNIT.EQ.6) GO TO 770
                WRITE (IUNIT,900) MEM,(RMS(J,MEM),J=4,7),BMCAP(MEM),
      1     DAMRAT(MEM)
  770          CONTINUE
C
               GO TO 800
  780          CONTINUE
C
C   SET DAMPING RATIOS TO 'APPROPIATE' VALUES FOR INITIAL TRIAL.
               DO 790 MODEA=1,NMODES
                  BETAM(MODEA)=DAMPIN
  790          CONTINUE
C
               ICOUNT=ICOUNT+1
               IF(ICOUNT.GT.25.AND.IUNIT.EQ.6) GO TO 800
```

```
            WRITE (IUNIT,840)
  800 CONTINUE
C
      ICOUNT=ICOUNT+1
      RETURN
  810 FORMAT('-','MODAL PARTICIPATION FACTOR',/)
  820 FORMAT(' ',5X,'MODE',I5,5X,F10.5,5X,F10.5)
  830 FORMAT('-',7X,'JOINT NO.',10X,'X-DISP(FT)',10X,'Y-DISP(FT)',7X,
     1       'ROTATION(RAD)')
  840 FORMAT('-',110('*'))
  850 FORMAT('-','ROOT MEAN SQUARE DISPLACEMENTS')
  860 FORMAT(' ',6X,I10,3F20.4)
  870 FORMAT('-','ROOT MEAN SQUARE FORCES')
  880 FORMAT(1H0,7X,'RSS BASE SHEAR =',F10.3,' KIPS')
  890 FORMAT('-',8X,'MN',10X,'AXIAL',10X,'SHEAR',11X,'BML',12X,'BMG',
     1       9X,'MOMENT',10X,'DAMAGE'/21X,'KIPS',12X,'KIPS',2(9X,
     2       '(K-FT)'),    8X,'CAPACITY',9X,'RATIO')
  900 FORMAT(' ',5X,I5,6F15.3)
      END
C
C=====================================================================
C
      SUBROUTINE SPECTR(ISPEC,DAMP,TN,AMAX,SA,WN,SABND,SVBND,SDBND)
C
C=====================================================================
C
C
C     ISPEC=1 IF SPECTRUM A IS USED
C          =2 IF SPECTRUM B IS USED
C          =3 IF SPECTRUM C IS USED
C          =4 IF NBC SPECTRUM IS USED
C     DAMP=DAMPING FACTOR (FRACTION OF CRITICAL DAMPING)
C     TN  =NATURAL PERIOD IN SECONDS
C     AMAX=MAXIMUM GROUND ACCELERATION (FRACTION OF G)
C     SA  =RESPONSE ACCELERATION (FRACTION OF G)
C     WN  =NATURAL FREQUENCY IN RADIANS PER SECOND.
C
      IF(ISPEC.EQ.2) GO TO 10
      IF(ISPEC.EQ.3) GO TO 60
       IF(ISPEC.EQ.4) GO TO 100
C
C     SPECTRUM A
C
      IF(TN.LT.0.15) SA=25.*AMAX*TN
      IF(TN.GE.0.15 .AND. TN.LT.0.4) SA=3.75*AMAX
      IF(TN.GT.0.4) SA=1.5*AMAX/TN
      GO TO 90
C
C     SPECTRUM B
C
```

```
10     CONTINUE
       IF(TN.LT.0.1875) GO TO 20
       IF(TN.LT.0.53333333) GO TO 30
       IF(TN.LT.1.6666667) GO TO 40
       IF(TN.LT.1.81666667) GO TO 50
       SA=2.*AMAX/(TN-0.75)
       GO TO 90
20     SA=20.*AMAX*TN
       GO TO 90
30     SA=3.75*AMAX
       GO TO 90
40     SA=2.*AMAX/TN
       GO TO 90
50     SA=1.875*AMAX
       GO TO 90
C
C      SPECTRUM C
C
60     CONTINUE
       IF(TN.LT.0.15) GO TO 70
       IF(TN.LT.0.38333333) GO TO 80
       SA=0.5*AMAX/(TN-0.25)
       GO TO 90
70     SA=25.*AMAX*TN
       GO TO 90
80     SA=3.75*AMAX
90     CONTINUE
       SA=SA*8./(6.+100.*DAMP)
       RETURN
C
C  NBC SPECTRUM
C
100     CONTINUE
        SV=40.0*AMAX
        SD=32.0*AMAX
        SACC=1.0*AMAX
C PRINT OUT A CAUTION NOTE SHOULD DAMPING BE LESS THAN 0.5%
        IF(DAMP.LT.0.005) WRITE(7,110)
110     FORMAT(' ','CAUTION-DAMPING LESS THAN 0.5%')
C
C  COMPUTE MULTIPLICATION FACTOR FOR ACCELERATION AT DESIRED DAMPING
        IF(DAMP.LE.0.02) AML=4.2+((0.02-DAMP)/0.015)*1.6
        IF(DAMP.GT..02.AND.DAMP.LE..05)AML=3.0+((.05-DAMP)/.03)*1.2
        IF(DAMP.GT.0.05.AND.DAMP.LE.0.1)AML=2.2+((0.1-DAMP)/0.05)*0.8
        IF(DAMP.GT.0.10) AML=1.0+((1.00-DAMP)/0.90)*1.2
C
C     COMPUTE MULTIPLICATION FACTOR FOR VELOCITY AT DESIRED DAMPING.
        IF(DAMP.LE.0.02) VML=2.5+((0.02-DAMP)/0.015)*0.8
        IF(DAMP.GT..02.AND.DAMP.LE..05)VML=2.0+((.05-DAMP)/.03)*0.5
        IF(DAMP.GT..05.AND.DAMP.LE.0.1)VML=1.7+((0.1-DAMP)/0.05)*0.3
```

```
      IF(DAMP.GT.0.10) VML=1.0+((1.00-DAMP)/0.90)*0.7
C
C     COMPUTE MULTIPLICATION FACTOR FOR DISPLACEMENT AT DESIRED DAMPING.
      IF(DAMP.LE.0.02) DML=2.5+((0.02-DAMP)/0.015)*0.5
      IF(DAMP.GT.0.02) DML=VML
C
C  COMPUTE BOUNDS USING DAMPING FACTORS COMPUTED ALREADY
      SDBND=SD*DML
      SABND=SACC*AML
      SVBND=SV*VML
C  COMPUTE WHICH IS THE APPROPIATE BOUND.
C  CONVERT FROM IN/SEC**2 TO FRACTION OF G BY DEVIDING BY 386.4
C
      SAATAP=SVBND*WN/386.4
      IF(SAATAP.GT.SABND) SA=SABND
      IF(SAATAP.GT.SABND) GO TO 120
      SDATCP=SVBND/WN
      IF(SDATCP.GT.SDBND) SA=SDBND*WN*WN/386.4
      IF(SDATCP.GT.SDBND) GO TO 120
C
C   IF HAVE NOT YET GONE TO STEP 180 THEN NATURAL FREQUENCY LIES ON
C VELOCITY BOUND.
C
      SA=SVBND*WN/386.4
C  SA IS RETURNED AS A FRACTION OF GRAVITY, G
C
120   RETURN
C
      END
C
C=================================================================
C
      SUBROUTINE SCHECK(S,NU,NB,IDIM,IUNIT,SRATIO)
C
C=================================================================
C
C  THIS SUBROUTINE CHECKS THAT ALL DIAGONAL STIFFNESS MATRIX
C  ELEMENTS ARE POSITIVE NUMBERS GREATER THAN ZERO.  IT ALSO DETERMINES
C  THE RATIO BETWEEN THE LARGEST AND SMALLEST MEMBERS ON THE DIAGONAL
C  THIS WILL GIVE SOME INDICATION AS TO THE CONDITIONING OF THE
C  STIFFNESS MATRIX
C  MATRIX
C
      REAL*8 S(IDIM)
      REAL*8 SMIN,SMAX,DIAG,RATIO
C
C
C   THE STIFFNESS MATRIX IS STORED AS A COLUMN VECTOR.  ONLY THE
C  THE LOWER TRIANGLE ELEMENTS BEING STORED (BY COLUMNS)
C  S(1) IS ON THE DIAGONAL AS IS S(1+NB),S(1+2*NB),ETC.
```

```
C  NB IS THE HALF BANDWIDTH OF THE STIFFNESS MATRIX
C
C  INITIALIZE THE LARGEST AND SMALLEST VALUES OF DIAGONAL (SMAX,SMIN)
C
      SMIN=1.0D45
      SMAX=-1.0D00
C
       DO 50 IDOF=1,NU
         IELEM=((IDOF-1)*NB)+1
         DIAG=S(IELEM)
C  COMPUTE IF DIAGONAL ELEMENT IS ZERO OR NEGATIVE
         IF(DIAG.NE.0.0D00) GO TO 20
         WRITE(7,10) IDOF
10       FORMAT(///' PROGRAM HALTED-A ZERO IS ON THE DIAGONAL OF STIFFNE
     1SSMATRIX',//'EXAMINE DEGREE OF FREEDOM ',I4)
          STOP
C
20      CONTINUE
         IF(DIAG.GT.0.0) GO TO 40
         WRITE(7,30) IDOF
30       FORMAT(///' PROGRAM HALTED-NEGATIVE ELEMENT ON DIAGONAL OF ',
     1          'STIFFNESS  MATRIX',//' EXAMINE DEGREE OF FREEDOM',I4)
         STOP
40        CONTINUE
C
C  DETERMINE IF THE DIAGONAL ELEMENT UNDER EXAMINATION IS THE LARGEST OR
C  SMALLEST OF THE DIAGONAL ELEMENTS.
         IF(DIAG.GT.SMAX) SMAX=DIAG
         IF(DIAG.LT.SMIN) SMIN=DIAG
C
50     CONTINUE
C
       WRITE(IUNIT,60)
60     FORMAT(/' ALL ELEMENTS OF MAIN DIAGONAL OF STIFFNESS MATRIX',
     1          ' ARE POSITIVE DEFINITE')
C
C  COMPUTE AND PRINT RATIO OF LARGEST TO SMALLEST DIAGONAL ELEMENTS
C
      RATIO=SMAX/SMIN
      SRATIO=SNGL(RATIO)
       WRITE(IUNIT,70) SRATIO
70     FORMAT(' ','RATIO OF LARGEST TO SMALLEST DIAGONAL STIFFNESS',
     1          'MATRIX ELEMENT IS',E10.3)
C
         RETURN
      END
End of File
```