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AUTOMATED DESIGN OF EARTHQUAKE RESISTANT MULTISTORY STEEL BUILDING FRAMES

by

Norman D. Walker, Jr.

A report on research sponsored by
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The design process as a whole is explored first, followed by a detailed investigation of the "frame-sizing" portion of this process. Static loading, in the form of dead/live load on the beams, and earthquake generated horizontal ground motion are considered in evaluating structural system response. Linear and approximate nonlinear analyses are employed.

Expressions describing the frame-sizing process are introduced. In addition an automating algorithm is presented. These procedures are then employed on two example problems which serve to develop insight into the design philosophy under study and into the operating characteristics of the proposed automated design procedure.

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ABSTRACT

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1. INTRODUCTION

Since the beginning of time, and with considerable vigor since the industrial revolution, engineers have sought to increase man's productivity through automation. Having been highly successful in this endeavor it is appropriate that engineers should seek to increase their own productivity through similar means. The engineer's task being one of thought rather than physical accomplishment, however, requires the use of high-speed computational facilities in order to mount a serious attempt at its automation. Hence, the introduction of the computer has facilitated substantial accomplishments in this area. There remains, of course, a considerable amount to do. Engineers everywhere are frequently confronted with highly repetitive operations, many of which could be automated, and most of which are not.

Nowhere is the presence of repetition more apparent than in design and in particular in structural design. This has been recognized for some time, with structural design proving to be one of the more fruitful areas for automation. Computerized attempts in this area data back to the 1950's with the subsequent proliferation of reports and papers far to numerious to mention. Several survey articles and books have resulted with [1] and [2] but two among many.

In structural design the primary emphasis of most efforts to date has been on the construction of automation procedures rather than on developing a definitive description of the tasks to be automated. The result has been the application of very mathematically sophisticated procedures to rather simple problems. This is not necessarily bad provided the simple problems suitably reflect the more complicated ones to which the automation procedures are to be eventually applied. In lieu of this what results is aptly labeled by Bellman [3] as an "inverted pyramid". Namely, volumes of material based on "mathematical models of ... dubious validity". As engineers, Pister [4] implores us to avoid this eventuality by maintaining a "proper balance in approaching structural analysis and design". This balance is best maintained by periodically reviewing the foundations upon which subsequent developments are to be built. Insofar as automated design is concerned this means a review of the tasks to be automated and their mathematical description. It is proposed that this effort represents an attempt in this direction, albeit a limited one, but nonetheless one in which the problem formulation assumes a more prominent position than the problem solution.

It should be noted that in the literature the terms "optimal" and "automated" are frequently used interchangeably. This will not be the case here. A distinction is maintained because the thrust of this effort is aimed at modeling and subsequently automating the design process. An optimal design does not necessarily result from such a procedure, only a usable design is assured. An optimal design procedure, if it is capable of starting from any initial design, constitutes a special type of automated design technique, namely, one which determines the optimal usable design.

1.1 Design Automation

In order to automate a design process a specific description of the procedure is necessary. For the work described herein a somewhat philosophical but useful definition of design has been formulated.

Design can be described as a complex collection of interrelated decision processes. This collection of decision processes has an entry point and an exit point, with the flow of decisions from entry to exit mobilized by a basic need, in this case the need for a structural support system. This concept is illustrated schematically in Figure 1.

Each individual decision process is composed of three parts: (i) a collection of usable options, called the option set, (ii) a criterion by which various options can be assessed, called the decision motivator, and (iii) a procedure by which the set of usable options can be explored to satisfy the decision motivator, called the option search. This decision process framework is illustrated in Figure 2. As a simple example of the process envision an individual who, while visiting friends, is offered a piece of candy from a box of candies. The option set is the collection of candies contained within the box. The decision motivator is the individuals personal taste preference. The option search mechanism is the individuals eyes, with which he explores the option set, coupled with his brain which helps him establish a taste value for each member of the option set. Perusing the option set and locating the member with the highest taste value establishes his choice.

Option sets can be divided into three categories: (i) sets composed of a continuous selection of usable choices, (ii) sets of discrete choices, and (iii) mixed sets of the preceding. Option sets containing a continuous array of possibilities clearly offer an infinite number of choices while discrete sets, in practice, always contain a finite number of alternatives. As an example of option sets in a structural context consider the problem of selecting a beam in a multistory building. If the designer is restricted to considering only wide flange rolled steel shapes, then he is dealing with a discrete option set with a finite number of alternatives. If, however, only fabricated members will do, then he is confronted with a continuous option set and an infinite number of possibilities. If he is free to chose either wide flange shapes or fabricated members, then he is dealing with a mixed option set.

Decision motivators are classifiable into two broad categories. The first category, labeled consignable motivators, consists of decision criteria which allow the assignment of numerical value, either scalar or vector, to each design possibility, thereby facilitating direct comparison of alternatives. The second category, referred to as subjective motivators, is composed of criteria for which no consistent, uniformly acceptable scale of value assignment can be devised. Weight, cost and reliability are good examples of consignable motivators for structural systems. For multistory buildings subjective motivators might include such items as esthetic quality, environmental impact, open space needs, etc.

Option search mechanisms can also be divided into two categories. The first category is composed of what are called discrete option search procedures. As the name implies these techniques are used to deal with decisions which have discrete option sets. These methods are generally combinatoric in approach and become quite inefficient when the number of members in the option set becomes large and the starting point for the search is far from the solution. The second category is composed of continuous option search mechanisms. These techniques are used to deal with option sets which are assumed to be continuous. The term "assumed" here is important because these procedures can be employed on decisions with discrete option sets, provided there is a sufficient number of members in the option set to warrant continuous modeling. Typically, continuous search techniques are more general in their applicability and for most problems more efficient in their operation than discrete search approaches.

From the preceding discussion it is clear that to automate design is to model the various decision processes involved, both individually and collectively. In structural design there is frequently an enormous number of decisions required, reflecting a wide diversity of motivators and option sets. To completely model all of these decision processes is in all likelihood impossible. It is preferable to isolate individual decisions or small groups of interrelated decisions and study then a few at a time. This format will be followed here. In restricting the investigation to individual or small groups of decisions it is implicitly assumed that these decisions can be uncoupled from the remaining body of decision processes. Intuitively it would seem that this is not true in general but that groups of highly coupled decisions could possibly be isolated so that this assumption is approximately satisfied.

As an example for the preceding discussion consider the subject to be explored here, namely the design of multistory steel building frames. A general schematic of the design of buildings is given in Figure 3. The process has been divided into distinct phases. The first phase, design concept formulation, in all likelihood involves decisions with subjective motivators and thus is not particularly susceptible to automation. If this is the case, very little if any coupling exists between the decisions involved in this phase and those of the remaining phases. If the sponsor's overriding concern is cost, however, this situation changes drastically with the design concept becoming heavily dependent on the decisions of subsequent phases.

The outcome of the first phase is almost always structural geometry. Frequently a primary material preference also results as well as a rough exterior wall design. Assuming a primary material is not selected, this becomes the second phase.

If the main concerns in primary material selection involve consignable motivators then automation of this decision is possible. The option set is discrete and may contain as few as three members, namely, the best concrete, steel and composite designs. Clearly a discrete option search mechanism is called for. Some work has been done on automating the design process from this level [5]. Assume here, however, that material selection results from the design concept formulation.

The next phase involves detailed design. From decisions made previously the frame material and geometry have been chosen so that frame design has been reduced to sizing. In addition the exterior wall design has been roughly specified. The coupling among decisions within this phase is fairly straightforward. The decisions involved in flooring, walls, lighting etc. in general do not depend on the frame and foundation size, however, the frame and foundation size depend directly on the remaining body of decisions in this phase. This decision coupling is accomodated by the imposition of dead weight on the frame and foundation, the amount of dead weight being a reflection of the choices made in the other decisions. Admittedly, this coupling model involves assumption, it does represent a very good approximation of reality, however. The framing and foundation decisions are typically uncoupled by a similar approach, again with a relatively accurate coupling model the result.

What is left of the design process after applying the previous developments has been roughly referred to as "frame sizing". It is important to note here that while some care has been taken to accommodate the coupling between "frame sizing" and the remaining body of decisions in the design process, what has resulted is nonetheless a suboptimization problem. While "frame sizing" constitutes an important part, it represents only a small portion of the overall design process. An enormous number of decisions still remain, however. Further problem confinements are, thus, necessary and are discussed in the following section.

1.2 Program Scope

The subject being examined here has been classified as the design of multistory steel building frames. As noted at the close of the previous subsection this portion of the overall design process encompasses a formidable number of decisions. For example still to be considered are such questions as: (i) should the frame be braced or unbraced, (ii) if the frame is to be braced, what kind of bracing should be used and where, (iii) what type of connection is to be used, rigid, semirigid or flexible and should they be bolted or welded, (iv) should fabricated members be employed, or available rolled shapes, (v) what type of steel should be used, etc. Note that all of these questions are to be answered in addition to deciding what size members, connections, welds, etc. are to be specified.

Clearly further problem restrictions are required in order to yield a project of manageable scope. To this end the class of problems to be considered is restricted to welded, unbraced moment resistant frames with members composed of standard rolled steel wide flange shapes of A36 steel. With this reduction the decisions which remain involve the selection of wide flange shapes for the beams and columns and the selection of connection rigidity.

As mentioned in the preceding subsection the coupling from the remaining body of "sizing" decisions is reflected through the application of dead load to the structure. The only additional loads on the structure which will be considered are live load and earthquake-generated horizontal ground motion.

Most of the concepts used in this work do not depend on frame geometry. In those instances where this is not true the discussion will be limited to single bay frames. A schematic of such a structure with impressed loads is given in Figure 4. Note that the dead/live load is uniformly distributed along the beams with torsional springs at the ends of the beams representing beam-column connections.

In reducing the structural system to that given above a concerted effort was made to develop a problem of tractable proportions and yet retain as much as possible the salient features of larger and more general structures of this type. Obviously only future studies will indicate the measure of success achieved in this regard.

At this point it is appropriate to introduce the concept of a design vector. In order to systematize the selection of wide flange members and connection rigidities it is necessary to associate descriptive variables with the option sets composed of these items. These variables must be sufficient in number to uniquely identify each member of all the option sets. Typically, once these variables are established they are arranged into a column matrix and referred to as the design vector. The design is thus complete when a design vector is specified. More will be said on this later, the concept being all that is necessary for the moment.

1.3 Report Outline

The basic philosophy which will be followed in the development of a design model for steel building frames is presented in Section 2. In Section 3 the option set description for wide flange members is developed, including the selection of descriptive variables for inclusion in the design vector. A similar development for connection option sets is given in Section 4. The motivator for the various decisions as well as the overall design process is presented in Section 5. Constraints which the public typically imposes on building designs are illustrated in Section 6. The culmination of the preceding six sections is given in Section 7 where a complete examination of the resultant design space is presented along with important implications about the design model. An option search mechanism is reviewed in Section 8 with examples therefrom discussed in Section 9. Section 10 presents an examination of the sensitivity of the design choice to various parameters involved in the system development. Some comments regarding the results of the effort are given in Section 11. All figures have been conveniently located in one place for quick reference and can be found following the main text.

2. DESIGN PHILOSOPHY

The first question to be confronted when embarking on an effort such as this is to which audience should it be addressed. For the author, a practical individual, the decision was easy - the design office. Not the present-day design office - some allowance for development time is essential - rather the "next generation" of design office is in mind. In attempting to provide automated procedures to the design community it is important to assess what the community will accept. To this end a look at present and expected future practices is necessary. Some very important consequences of the choice of program direction result, the first being the selection of a decision motivator.

There is a variety of potential decision motivators available for multistory building design. Reliability, for example, has been used extensively in this regard [6]. Serviceability safety and cost are further possibilities. Historically, however, with the major exception of cost, society has for its own protection chosen to legislate minimally acceptable levels for most of these quantities. Designers have, thus, been left to minimizing costs within legislated design restrictions. Considerable evidence exists at present to indicate that this practice will continue. Hence, historical precedence will be followed and cost chosen as the decision motivator for this effort. Note that legislated design limitations are usually packaged under the name of building codes and will be referred to here as system constraints.

The next major consequence of program direction deals with the loading conditions, in particular the earthquake loading. There are two avenues of approach in dealing with earthquake loads, the probabilistic and the deterministic. The design community has in general shied away from using the probabilistic approach for two reasons. First, the probabilistic format is considerably more complex, defying intuition. Second, the techniques for probabilistic assessment of structural adequacy with regard to earthquake loading are not yet developed enough for standard design office use. It is fair to say that these procedures are still years away from design-level application. Hence, the design community by and large relies on deterministic techniques, as will be done here.

The last major implication of the design office emphasis of this effort lies in the assumed availability of analytical tools. Design practitioners generally utilize linear procedures for structural analysis because of their ease of application and relatively low cost. Nonlinear techniques are typically quite expensive. Expecting this situation to prevail for some time to come, the analytical tools used in the sequel are restricted to linear procedures.

With these developments in hand a closer look at the major components of building frame design is now undertaken with the intent of developing a complete outline of the design philosophy to be employed here.

The following discussion is divided into three parts, dealing in further detail with the decision motivator, the system constraints and the assumed analytical tools.

2.1 Decision Motivator

The costs associated with a multistory building quite naturally fall into two categories: (i) the cost of designing and erecting the building, referred to as construction costs, and (ii) the cost of maintaining a building including the repair of damage due to structural overload. To date, project sponsors and design firms have been content to develop building systems with construction cost minimization the primary concern. It has been frequently suggested, however, that a more rational approach is to design with the minimization of lifetime cost (LC) the principal consideration. Here LC represents the construction cost plus the maintenance cost. When automated design procedures become feasible for large structural systems, it seems very likely that the LC approach will be adopted as the most logical one to utilize. For this reason LC is employed as the decision motivator for this effort.

There are two ways of approaching the formulation of the above decision motivator. The first is to develop a complete estimate including all construction and maintenance costs. This approach while quite valid is far from expedient. Many construction and maintenance costs are independent of the design vector introduced in Section 1. That is, changes in the design vector have little or no effect on these costs. Costs which are independent of the design vector do not participate in the design process. They are constant and retain the same value regardless of the choices made. Independent costs, thus, have no effect on the ultimate design selection. Hence, independent costs do not need to be taken into account in the formulation of the decision motivator. Ignoring independent costs constitutes the second approach to decision motivator formulation. It is clearly the more expedient and consequently will be employed here.

It is important to keep in mind that the set of independent costs depends directly on the contents of the design vector. Different types of design vectors foster different sets of independent costs. If the complete (global) design problem is being confronted then very few, if any, costs are independent, in which case the above approach is of little use.

Reviewing the costs related to construction [7] and eliminating independent and mildly dependent costs results in a list of construction costs strongly related to the chosen design vector. They are

(i) the cost of the structural steel shapes from which the beams and columns are selected

(ii) the cost of the beam-column connections including structural steel and welding

(iii) the cost of transporting the structural steel from the supplier or fabricator to the site

(iv) the cost of field painting the assembled structural frame

(v) the cost of overhead and profit.

Models of these costs based on the components of the design vector will be developed in subsequent sections. In the meantime this assembled list will provide guidance in option set development.

Note that erection costs are not included in the above list of construction expenses. It is assumed herein that moderate variations in individual member weight have little effect on the erection process.

Thus, erection costs are independent of the design vector. While this appears to be a good assumption with regard to steel structures it is obviously not applicable in all situations, such as in the case of concrete frames for example.

A review of maintenance costs for multistory buildings quickly reveals that the only design vector dependent cost is that of building repair due to structural overload. For buildings located in seismically active regions the major contributor is earthquake overload. For this effort this is assumed to be the only contributor. In order to develop these costs two items require further attention:

(i) it will be necessary to construct building damage models which are based on structural response parameters

(ii) an expected earthquake profile for the life of the building will have to be identified.

With these in hand an estimated LC due to earthquake damage can be constructed. The details of this development are presented in the subsequent sections.

Observe here that the inflationary aspects of the economy, alternative investment possibilities, prevailing interest rates (cost of money), etc., will not be included in damage estimation. Such an accommodation is obviously necessary for a complete model. A constant dollar, unencumbered by the numerous economic considerations regarding its availability and expenditure, is assumed here, however, to avoid becoming embroiled in a subject somewhat remote from the main trust of this effort.

2.2 System Constraints

As noted earlier the loading conditions which are to be considered are constant distributed dead/live load on the beams and earthquake-generated horizontal ground motion.

As far as the above static loading is concerned the more prominent of the numerous building code restrictions, and the only ones to be considered here are:

(i) internal member forces are to be limited to a value less than yield level for the member

(ii) midspan beam displacements must be less than a specified amount

(iii) axial forces imposed on the columns must not exceed a specified fraction of the load that would produce buckling of the column.

These constraints will be developed more fully in a later section. For the moment, however, they will serve to guide the option set modeling which follows this section.

With regard to the dynamic (i.e., earthquake) loading present practice is very simple. In order to simulate the action of an earthquake, static lateral forces are imposed over the height of a structure. These forces are imposed in addition to the beam dead/live loading, with a reduction in live load frequently allowed. The structure is required to satisfy roughly the same constraints imposed for the static loading but normally with an increase in the allowable stresses. There are two major objections to this approach [8, 9, 12]. First it is felt that the lateral forces presently specified represent only small earthquakes and fall substantially short of the actual earthquake forces a structure in a seismically active region would likely be subjected to at some point in its lifetime. Second, present design requirements do not call for specified levels of ductility within a structure even though inelastic response is certain for moderate and strong earthquakes.

An alternative to the above approach which has been frequently suggested [9, 10, 11, 12, 13] is based on a dual design criterion. It can be roughly stated as follows [9]:

(i) the structure should respond elastically to a moderate earthquake of an intensity reasonably anticipated within its lifetime

(ii) the structure may yield significantly but must avoid collapse during a maximum credible earthquake.

This criterion has gained considerable acceptance and thus will be adopted here. The necessary details of the criterion will be developed in a later section. What is important to note here in the application of this design guide is the necessity of avoiding collapse. There are several procedures presently available which purport to do this. The most popular of these is the so-called strong column-weak girder design philosophy. Under this procedure a structure is designed so that during a strong earthquake inelastic activity is confined as much as possible to the beams rather than the columns. The basic belief is that if the columns remain elastic or nearly so then collapse is very unlikely.

It is generally accepted that complete avoidance of inelastic activity in the columns is not possible. Plastic deformation sometimes occurs in first story columns, due in part to the large loads and in part to the relative fixity of the column bases. In addition plastic deformation frequently takes place in the upper story columns due to stress wave reflection ("whiplash"). Inelastic activity in the upper story columns does not carry the gravity of similar activity in lower stories, however, since the axial loads are much less, leaving considerable ductile capacity in these members.

Inelastic activity in structures is typically cited in terms of ductility ratios. For this study the ductility ratio is defined as the maximum total end rotation of a member divided by its elastic limit end rotation [14]. To determine if a building satisfies the strong column-weak girder design philosophy some procedure for computing the ductility ratios throughout the structure will be necessary.

In addition to the above developments it should be noted that the present trend in the design community is away from static force simulation of earthquakes and towards the use of simple dynamic analysis [10, 11, 13]. In keeping with this trend simple dynamic analysis procedures are employed in this effort.

2.3 Analytical Tools

No discussion of design philosophy would be complete without an assessment of available analytical tools. As noted earlier only linear procedures will be employed here. In addition dynamic rather than static analysis will be used for earthquake loading. Thus, this discussion logically falls into two categories, linear static and linear dynamic techniques.

For the assessment of structural adequacy in the presence of static loads standard matrix analysis procedures are employed. In the case of distributed loading the structural problem is separated into two subproblems. First, the structure is subjected to the distributed loading and sufficient joint forces to rigidly fix all of the external degrees of freedom. The first subproblem thus amounts to solving a set of fixed-end, distributed loading problems for the beams. Second, the structure is loaded with only the negative of the fixing forces which were used in the first subproblem. This second subproblem is solved using matrix analysis techniques. The results of the two subproblems must be added to obtain the complete solution. In this effort axial and shear displacements are not taken into account, nor are P-Δ and beam-column effects.

For the dynamic analysis the mass of the system is assumed to be collected at the story levels (lumped-mass). In addition Caughy damping [15] is utilized. This along with the assumption of linearity allows mode superposition techniques to be employed. For earthquake loading an estimate of the maximum modal responses can be obtained through the use of earthquake response spectra. To estimate the response of the structure itself the modal results are combined using the familiar root-sum-square (rss) method.

For this study a variant of the Newmark-Hall response spectra [16] is adopted. The Newmark-Hall procedure for elastic response will be followed explicitly with the exception of the "acceleration transition" region. This region of the spectra represents the high frequency (low period) response of structures and is seldom significant in practice. In this portion of the design spectra it will be assumed that the response acceleration remains constant and does not undergo a transition to the ground acceleration as specified in [16].

The important thing to note about the Newmark-Hall procedure is that if necessary the ground motion can be completely characterized by the specification of a single parameter. In this effort peak ground acceleration is employed in this capacity. In addition the selection of one more parameter, damping ratio, facilitates the construction of a complete response spectrum for an elastic structure. These characteristics will be used to good advantage in the sequel.

As noted in the preceding subsection it will be necessary to compute the ductility demands of a structure in order to assess its ability to withstand collapse. The only technique presently available for doing this on the basis of a linear analysis is the so called "ductility factor" method. Using this procedure an approximate ductility ratio, called the ductility factor, can be computed by dividing the maximum member moment by the member moment capacity (plastic moment). This procedure, while approximate, appears to have some merit [14] and since it is the only one available will be employed herein. To facilitate its use a moment-rotation relationship is assumed for wide flange members and is given in Figure 5. This is a fairly standard representation with the plateau corresponding to the member plastic moment M_p. In the absence of axial load the plastic moment is equal to the yield stress of the member material times the member plastic section modulus.

Before moving on to option set modeling it should be noted that almost all of the above material on dynamic analysis can be found in considerable detail in [12].

3. BEAM AND COLUMN OPTION SETS

Beam and column option sets are assumed to be composed of the collection of all Regular Series wide flange rolled steel shapes as identified by the American Institute of Steel Construction (AISC) [17]. This set is clearly discrete. A sufficient number of members exist within this set, however, to warrant continuous modeling. As noted in the introduction, continuous option search mechanisms are more generally applicable and typically more efficient than discrete approaches, hence, continuous modeling is attempted here.

The primary difficulty in modeling discrete sets using continuous functions is making the selection of an appropriate set of descriptive variables. The essential task of these variables is to uniquely label each individual member of the option set. Quite clearly it would be beneficial to select as few variables as possible in order to keep the magnitude of the eventual design problem to a minimum. In addition to this the physical boundaries of the option set and the various properties of the members within the set have to be developed in terms of the chosen descriptive variables. Thus, these variables should be selected so as to facilitate a reasonably accurate and smooth functional description of the option set boundaries and member properties.

Descriptive variables are typically selected from member physical dimensions and/or derived properties. For wide flange sections the physical dimensions consist of the member depth d, width w, flange thickness t_f and web thickness t_w. The derived properties are the strong axis moment of inertia I, elastic section modulus S, plastic section modulus Z, cross sectional area A and strong axis radius of gyration r. Weak axis properties may also be included but are not particularly useful in planar frame design. Obviously the maximum number of descriptive variables necessary is four: d, w, t_f, t_w. If nothing else, selection of these variables leads to very accurate representations of the other properties of the member. Reduction of the number of variables is possible, however. This reduction is effected in two ways. First, through the development of empirical relationships between the available variables and second, through the use of suboptimization. A price is paid for this reduction, of course. Use of empirical relationships sacrifices accuracy in member property representations and suboptimization results in a smaller option set.

As an example of an empirical reduction consider the approximate relationship $t_w \cong 0.61\ t_f$. This equation represents the mean for all compact wide flange sections. It is very accurate for some members and not so accurate for a few. It does facilitate a reduction of the descriptive variable set to three, however.

Are further empirical reductions possible? In terms of uniquely specifying members of the option set the answer is yes. AISC for example labels wide flange shapes by member depth versus weight per unit foot, i.e. area times a constant. Thus, d and A form a potential descriptive variable pair. Since d and A constitute a viable pair, it is fairly obvious that any two independent quantities from the set d, w, t_w, t_f, I, S, Z, A and r are potential descriptive variables. Note that t_w and t_f can not serve in this capacity since as shown earlier they are not independent (in an average sense). Likewise, as will be seen later, S and Z or d and r can not function in this manner.

Of those pairs which can serve as descriptive variables it remains to be seen if any admit the formulation of smooth, accurate functional representations of the option set boundaries and member properties. In order to answer this question such representations must be developed. To this end I and d are selected as a test descriptive variable pair. The moment of inertia is used because wide flange sections are rolled specifically to produce a continuous selection of I, a very desirable characteristic when attempting to develop empirical relationships. This continuous array of I is obtained by rolling the members on several different plateaus of member depth. Thus, in conjunction with I, d would seem to be a logical choice if an even dispersion of members throughout the option set is to be obtained.

In order to facilitate the development of accurate modeling the collection of wide flange sections is split into two option sets, one for columns and one for beams. In addition the range of moment of inertia included with these sets is restricted to include only those members which are likely to be of use for the structures considered here. For columns appropriate moment of inertia limits are $200 \text{ in}^4 \leq I \leq 1500 \text{ in}^4$ and for beams $180 \text{ in}^4 \leq I \leq 2500 \text{ in}^4$. The resultant option sets are plotted in Figures 6 and 7. The dashed lines in these figures represent the above modeling bounds whereas the solid lines represent the bounds on availability within the modeling limits. Each of the dots signifies a Regular Series wide flange section. The circled dots indicate economy sections and the triangled dots the antithesis, with the bounds on availability being least square power curve fits to these two sets of points. The long-short dashed lines are extrapolations of the availability bounds beyond the modeling limits. As can be seen, in general, the extrapolations are not very good. This is particularly true in the case of the column bounds wherein the extrapolations cross and, hence, become quite meaningless. The empirical equations for the availability bounds are

$$2.94\ I^{0.220} \le d \le 2.66\ I^{0.287} \tag{3.1}$$

for the beams and

$$2.22\ I^{0.253} \le d \le \begin{cases} 0.91\ I^{0.447} & \text{for } I < 429\ \text{in}^4 \\ 10.5\ I^{0.0436} & \text{for } I \ge 429\ \text{in}^4 \end{cases} \tag{3.2}$$

for the columns.

As part of a complete model empirical relationships are needed for all other required member properties. For example, with beams, A and Z may be needed and for columns A, Z and r. To develop these relations a few observations are in order. First, note that r represents the location of a lumped cross sectional mass for an equivalent I: $I = \int y^2\, dA = A\, r^2$, where y is the distance from the neutral axis. Since most of the bending stiffness in wide flange sections is supplied by the flanges and the flange radius of gyration is roughly d/2, a fairly strong correlation between r and d/2 is expected. A plot of r versus d/2 for column sections of interest is presented in Figure 8. The solid line is the least squares curve fit given by $r \cong 0.39\ d^{1.04}$. A similar relationship exists for beams and is given by $r \cong 0.52\ d^{0.92}$. Since $A = I/r^2$ the above equations also yield relationships for A. For Z note that $Z \sim S \sim I/d$. A mean value computation results in $Z \cong 2.25\ I/d$.

From Figures 6, 7 and 8 it is clear that smooth, accurate functional representations of the option set bounds and member properties have been obtained on the basis of the descriptive variable pair I and d. A viable option set for use with continuous option search procedures is thus available.

To reduce the number of independent variables to one, additional parameterizations are possible. If done on the basis of the physical dimensions and derived properties for wide flange sections listed earlier these developments result in models which are very inaccurate and represent only mean values rather than estimates of actual value. An alternative approach is through the use of contrived parameters. A definite improvement in accuracy ensues; however, the resultant modeling functions are extremely irregular and relativly incompatible with conventional option search procedures. Instead, what most researchers do is resort to suboptimization in order to obtain single variable representations of option sets.

As far as wide flange sections are concerned the usual suboptimization ploy is to restrict the option set to those members which result from minimizing weight for constant elastic section modulus. The resultant member set is composed of the AISC economy sections. These are the circled points in Figures 6 and 7. Clearly a single variable representation is now possible in terms of I. The primary sacrifice is a rather sizable reduction in the option set. Using this reduced option set involves a significant assumption. Namely, it is assumed that an option search without suboptimization results in a choice which lies within the reduced option set. Thus, the same choice would result from an option search with suboptimization. The best way to examine the validity of this assumption is not to employ it initially. Hence, suboptimization is not utilized. Instead a two variable representation of the option sets is incorporated.

The member properties of primary interest herein are I and Z, I because it is the principal measure of member stiffness and Z because it indicates the limit of this stiffness, i.e. the moment capacity. Hence, I and Z are selected as the descriptive variables. Using the equation $d = 2.25\ I/Z$, option set bounds on Z are established via equations (3.1) and (3.2). They are

$$0.85\ I^{0.713} \leq Z \leq 0.77\ I^{0.78} \tag{3.3}$$

for beams and

$$1.01\ I^{0.747} \geq Z \geq \begin{cases} 2.47\ I^{0.553} & \text{for } I < 429 \\ 0.21\ I^{0.956} & \text{for } I \geq 429 \end{cases} \tag{3.4}$$

for columns. For member area a fairly good relationship for both beams and columns is $A = 1.1\ Z^2/I$. As noted previously $r = \sqrt{I/A}$. The remaining property which is required in the sequel is member width. The width is needed to compute the member surface area for painting cost estimates. The development of an empirical equation for this parameter is not exactly straightforward. It can be obtained through an iterative application of improving approximations to I using the equation

$$I = wtd^2/2 + td^3/20 + wdt^2 + 2wt^3/3$$

along with

$$A = 2\ wt + 0.6\ td - 1.2\ t^2$$

where $t = t_f$ and $t_w = 0.6t$. Retaining only the significant terms, after two iterations the above process results in

$$w = \frac{2.35\ I/Z}{2.09 - 0.812\ Z^4/I^3} \tag{3.5}$$

This equation is not extremely accurate. It is to be used only in computing painting costs, however, and is probably sufficiently accurate for this purpose. Painting costs represent only a very small part of the overall LC of a building structure so that significant errors in this quantity are tolerable. More will be said on this in a later section.

This completes the specification of the option sets for beams and columns composed of wide flange members. The modeling developed for these sets is valid only over the limited selections which are specifically being modeled. Extrapolation of these models beyond these sets is not advisable and should be done only with considerable caution.

4. CONNECTION OPTION SET

Beam-column connections used in steel construction are generally classified according to their rotational characteristics as rigid, semirigid and flexible (simple) [18, 19, 20]. The degree of rigidity of a connection is defined as the ratio of the beam end moment developed with the connection in place, to the beam end moment developed with a fully rigid connection under the same conditions [21, 22, 23]. In terms of this definition of rigidity the above connection classifications are frequently defined as follows [21, 22, 23]: rigid connection are those which develop 90% rigidity or more, flexible connections produce 20% rigidity or less and semirigid connections constitute everything in-between.

Present design practice calls for rigid connections in most multistory building frames in seismically active regions. This has not always been the case, however. At the turn of the century most steel buildings employed semirigid connections, which as noted in [24] were found to perform quite satisfactorily during the San Francisco earthquake of 1906. Choice of connections would thus seem to offer a viable avenue of design modification and additional components for the design vector. A complete examination of the role of connection rigidity in earthquake design is of course not possible in light of the limited class of frames being considered here. Only a preliminary assessment of the role of connection rigidity in unbraced, lateral force resistant frames can be made. Nonetheless the connection modeling which follows in this and subsequent sections is generally applicable and transcends the limited question to which it is being addressed.

Use of the strong column-weak girder design philosophy imposes a major restriction on the selection of a beam-column connection. In order to tap the available ductility of beams as is required by this approach it is necessary that the chosen connections be able to develop the full moment capacity of these beams. There appears to be only one connection capable of accomplishing this while still offering some degree of flexibility and that is the welded top and bottom plate connection shown in Figure 9. This connection has been tested extensively and is accepted by AISC for semirigid framing. Even this connection is limited, however.

To facilitate a discussion of the welded plate connection some definitions are in order. An overhead view of a top plate is shown in Figure 10. Note that this top plate is detailed differently from that shown in Figure 9. The unwelded length of the plate is labeled L' and the welded length L'_w with A' signifying the plate cross-sectional area. The moment-rotation behavior of this connection can be represented as shown in Figure 11 [18, 25]. As given in the figure, k is the initial stiffness of the connection and M_{pc} is the connection plastic moment.

An analysis of this connection reveals that [18, 23, 25]

$$A' = M_{pc}/(\sigma_{yc}d) \qquad (4.1)$$

and

$$k = A'Ed^2/(2L') \qquad (4.2)$$

where E and σ_{yc} are the modulus of elasticity and yield stress of the connection steel with d the beam depth. Note that the connection center of rotation is assumed here to be located midway between the top and bottom plates. Since the welded plate connection must be capable of developing the plastic moment of the beam to which it is attached, it is necessary that $M_{pc} \geq M_{pb}$ where M_{pb} is the beam plastic moment. Assuming for simplicity $M_{pc} = M_{pb}$ then equation (4.1) becomes

$$A' = M_{pb}/(\sigma_{yc} d). \qquad (4.3)$$

The rigidity of the welded plate connection is given by [25]

$$R = \frac{1}{(4IL'/A'd^2L) + 1} \qquad (4.4)$$

where L is the beam length and I its moment of inertia. Using equation (4.3) and the associated beam property models from Section 3 in (4.4) yields

$$R = \frac{1}{(1.78\ \sigma_{yc} L'/\sigma_{yb} L) + 1} \qquad (4.5)$$

where σ_{yb} is the yield stress of the beam material. Only σ_{yc} and L' are available for adjusting the connection rigidity since, normally, $\sigma_{yb} L$ is fixed through other considerations. Full rigidity, that is $R = 1$, is easily obtained by taking $L' = 0$; it is semirigidity which is difficult to provide. Selecting σ_{yc} as large as possible is a step in the right direction. For example, of the structural steels A514 appears to be a good choice for connection plates since it offers a high yield strength, is weldable and relatively available. Having selected σ_{yc}, L' becomes the sole device for adjusting connection rigidity. There are limitations on L', however. Through earthquake activity the connections will likely be subjected to load reversals. Thus, both the top and bottom plates can expect to see compressive stresses and should probably be protected against buckling. Unless some means of preventing buckling is provided, a buckling constraint must be placed on the connection plates. For this purpose the limitation $L'/t \leq 30$ is suggested [18], where t is the plate thickness.

To gain a rough idea of the minimum rigidities obtainable using the welded plate connection the preceding analysis is completed with the help of several simplifying assumptions. Let the plates be made from square stock A514 steel, the beams from A36 steel with a length of 300 in., then equation (4.5) becomes

$$R = 1/(0.187\sqrt{A} + 1) \qquad (4.6)$$

where A is the beam area. A table of rigidity versus beam area is given in Figure 12. The areas listed in this table cover most commonly used beams. As can be seen the minimum rigidities range from about 50 to 65%. If a connection rigidity less than these values is required then clearly it must come at the expense of the connection moment capacity since according to equation (4.4) the only alternative is to reduce A'. For earthquake resistant structures, however, this range of connection rigidity is probably more than adequate.

The development of an option set model for the welded plate connection is quite straightforward. The set is continuous and in the general case requires two descriptive variables. Candidates for this assignment are A', L', k and M_{pc}. Since M_{pc} is specified through strong column - weak girder design requirements only a single descriptive variable is needed. As far as option set constraints are concerned, L' is probably the best choice. For analysis purposes, however, k is a better choice and is used in the sequel for this reason. From the above developments it is apparent that only a lower bound on k exists. Practically speaking, of course, this is not true. No "real world" connection is infinitely stiff, hence, an upper bound also exists. Rigid joints are frequently assumed in analysis, however, so no upper bound on k is imposed herein. In addition no lower bound is imposed since it seems highly unlikely that the minimum connection rigidities stated above will be breached by earthquake resistant structures.

This completes the specification of the option sets. The next item to be addressed is the decision motivator.

5. DECISION MOTIVATOR

As noted previously the decision motivator used in this study is lifetime cost (LC) including both the construction and maintenance cycles of the building structure. The costs which are to be modeled for inclusion in this motivator are restricted to those which are strongly dependent on the design vector. This vector, which has been referred to only figuratively up to this point, can now be clearly identified. It is composed of the moments of inertia and plastic section moduli of each of the beams and columns in the building frame along with the beam-column connection stiffnesses of this frame. The cost models developed in this section are constructed on the basis of the components within this design vector so that with each vector there is associated a unique LC.

The discussion of this section has been broken into two parts, one dealing with construction costs and the other with maintenance costs.

5.1 Construction Costs

The design vector dependent construction costs were identified in section 2.1. The following development is subdivided according to the categories given in that section: (i) structural member costs, (ii) connection costs, (iii) the cost of field painting, and (iv) the cost of overhead and profit.

5.1.1 Structural Members

There are generally three types of charges assessed on wide flange sections as purchased from a rolling mill [7]: (i) a base price which is levied on the weight of steel purchased, (ii) a size extra charge based on the shape of the purchased members, and (iii) a quantity extra charge assessed on the amount of material procured. Since each of these charges is based on a different aspect of the sections purchased, separate cost models are needed. Thus, in the subsequent discussion each charge is dealt with individually with cost models developed accordingly.

The first item listed above is the base charge which as noted is assessed on the basis of the weight of steel purchased. If C_{SM} is the price of the steel of which the members are composed then the cost of a single member is $C_{SM}\, AL\, \gamma$ where A and L are member area and length with γ the density of steel. The total cost of steel for all the wide flange members in a frame would, thus, be

$$\text{Total Cost} = C_{SM}\, \gamma\, \Sigma\, A_i L_i \tag{5.1}$$

where the summation is over all the members in the frame with the subscript i signifying individual member properties. Note C_{SM} is typically quoted in dollars per cwt (100 lbs).

The size extra charge is assessed on the basis of the cross sectional configuration (size) of a member and is also cited in dollars per cwt. A representative list [7] of size extra charges for wide flange sections is given in Fig. 13. In order to model this charge a relationship between the costs quoted in Fig. 13 and the descriptive variables of the associated members must be developed. Of the various combinations available the correlation between member area and size extra charge appears to be as good as any. A plot of this combination for the sections of interest is given in Fig. 14. As can be seen the correlation is not outstanding. However, a rough trend in size extra charge as a function of member area is clearly perceivable. It is equally clear that a model of this trend is all that is possible here since it is obvious that a continuous function cannot capture an accurate representation of the points plotted in Fig. 14. The curve in the figure represents a least squares fitted power function given by $0.916\ A^{-0.21}$. Thus, the size extra charge for a single member is $0.916\ A^{-0.21}\ AL\ \gamma = 0.916\ A^{0.79}\ L\ \gamma$. The total amount for an entire frame is therefore

$$\text{Total Cost} = 0.916\ \gamma\ \Sigma\ A_i^{0.79}\ L_i. \tag{5.2}$$

The final assessment on wide flange members is the quantity extra charge. This levy is determined by the weight of the total amount of an individual section type procured at one time for one mode of shipment to one destination. A sample quantity extra charge schedule [7] is given in Fig. 15. As can be seen this charge is inversely proportional to the weight of steel purchased. For wide flange members used in multistory buildings this charge is generally avoided since a significant amount of member duplication is typical in these types of structures. Hence, this charge is not accounted for in this study.

While transportation costs are not strictly a part of the member cost, because they are generally based on member weight they will be accommodated here. Let C_{TS} be the cost of transporting steel in dollars per cwt. Then this charge is best accounted for by simply adding it on to the cost of steel. Thus, $C'_{SM} = C_{SM} + C_{TS}$ where C'_{SM} is a modified price of steel for the members.

5.1.2 Connections

There are two costs associated with the installation of welded plate beam-column connections. First, there is the cost of the steel in the plates and, second, there is the cost of welding the plates. There are numerous qualifications which must be made in order to develop these costs; hence, the resultant models are of limited applicability. Connection costs are small in comparison to the price of the overall frame however; thus, the models developed here are probably adequate for the purposes of this study.

Let C_{SC} represent the cost of the steel from which the connection plates have been fashioned. Then the cost for each connection is given by $C_{SC}\ A'\gamma\ (2L' + L'_{WT} + L'_{WB})$ where L'_{WT} and L'_{WB} are the lengths of top plate and bottom plate which are welded to the beam. Since A' and L' have already been developed in section 4, it remains to specify L'_{WT} and L'_{WB}. To this end assume that the four primary legs of the fillet weld shown on the top plate in Fig. 10 are equal in length and that the short weld sections on the end of the plate are negligible, then $L'_{WT} = \ell_w/4$ where ℓ_w is the length of weld required to develop the plastic moment of the connection. From Fig. 9 it can be seen that the bottom plate is welded only along the outside edges of the bottom flange of the beam, hence, $L'_{WB} = \ell_w/2$.

To compute ℓ_w the electrode used to form the weld and the fillet weld size must be specified. Since A514 steel is anticipated as a plate material the luxury of E110 electrode is assumed along with a 3/8 in. fillet. This produces a fillet weld strength of 8.9 kips/in [20]. Thus, $\ell_w = M_{pb}/8.9 = 4.04\ Z/d$ since A36 steel is being used for the beams. Using this and the equations developed in sections 3 and 4 results in a cost estimate of $0.36\ C_{SC}\ \gamma\ Z^2$ $[(10800/k) + 0.6\ (Z/I)^2]$ for plate steel per connection where a modulus of elasticity of $30(10^3)$ ksi is assumed for structural steel. This cost has to be doubled, since there are two connections with each beam, and summed over all beam members to obtain a total cost for the connection plate steel of

$$\text{Total Cost} = 0.72\ C_{SC}\ \gamma \sum_{\text{beams}} Z_i^2\ [(10800/k_i) + 0.6\ (Z_i/I_i)^2]. \qquad (5.3)$$

Transportation costs are treated as they were for structural members and simply added into the cost of steel with $C_{SC}' = C_{SC} + C_{TS}$.

Welding costs originate from a variety of sources including labor, overhead, electricity and down time in addition to the cost of electrodes. Rather than model each of these sources individually it is more expedient to lump them together and quote welding charges in terms of dollars per pound of metal deposited. A welding cost schedule constructed on this basis is given in [7] with charges estimated in terms of electrode size. Thus, to compute the welding cost for a particular connection the amount of weld metal which must be deposited to complete the weld has to be established. This computation logically falls into two parts, one for the butt weld at the plate-column interface and one for the fillet weld along the plate-beam interface.

For the butt welds assume there is a 3/16 in. root gap between the end of the plates and the column and that the plates have been leveled to 45°. For the top plate assume square stock so that the plate thickness is $\sqrt{A'}$. Then the volume of metal contained in the butt weld for the top plate is $A'(3/16 + \sqrt{A'}/2)$. For the bottom plate assume a 10 in. width to insure overhang beyond the beam flange, then the volume of metal in the butt weld for the bottom plate is $A'(3/16 + A'/20)$. The cost of the butt welds for the two connections per beam is thus $C_{SW}\ \gamma\ A'(0.75 + 0.1\ A' + \sqrt{A'})$ where C_{SW} is the cost per pound of weld metal in place.

To compute the cost of the fillet welds assume they are built up 10% over specification for safety [23]. Then the cost of the fillet welds per beam is 2.42 $C_{SW}\ \gamma\ t^2\ \ell_w$ where t is the specified weld size.

Summing up the welding costs over the entire structure and substituting in the appropriate equations from this and previous sections results in a total welding cost estimate of

$$\text{Total Cost} = C_{SW}\ \gamma \sum_{\text{beams}} \frac{Z_i^2}{I_i} \left(0.73 + 0.0026\ \frac{Z_i^2}{I_i} + 0.064\ \frac{Z_i}{I_i^{0.5}}\right). \tag{5.4}$$

5.1.3 Structural Painting

Structural paint is typically applied in two coats using a spray gun. Sometimes paint is applied by hand if local codes prohibit the use of spray. Obviously the painting costs depend strongly on which method is utilized. Spray gun application is assumed here.

The cost of painting depends on two items, the cost of labor and the cost of paint. A labor cost is obtained by dividing the prevailing labor rate (\$/hr) by the rate at which the paint can be spread (in^2/hr) to obtain labor cost, C_{PL}, in dollars per square inch. An estimate of the cost of paint is arrived at by dividing the price of paint (\$/gal) by the spread rate (in^2/gal) and multiplying by the number of coats to obtain a cost of paint, C_P, in dollars per square inch. The complete cost of painting is thus given by $C_{PT} = C_P + C_{PL}$ and has the dimensions of dollars per square inch.

To compute the area which must be covered assume that the surface area of wide flange members is given by (4w + 2d)L. The total painting cost for the structure is therefore

$$\text{Total Cost} = C_{PT} \sum (4w_i + 2d_i)L_i. \tag{5.5}$$

5.1.4 Miscellaneous

If equations (5.1) through (5.5) are added together then what is obtained is essentially an estimate of the construction cost of the structural frame. The frame typically represents about 10-15% of the overall project cost. Assessed on top of this are overhead and profit rates. These are usually charged as a percentage of the overall project cost and thus can be represented as multiplicative constants. Overhead and profit each run somewhere in the neighborhood of 5-15% of the project cost.

5.2 Damage Costs

The modeling of damage costs due to earthquake overload is a particularly difficult task. The principal source of difficulty lies in the stochastic nature of earthquake loading. Two different earthquakes of equal magnitude can produce strikingly different results in terms of building damage. Loading is not the only source of difficulty, however. At present there exists an enormous diversity in construction materials, design practices and erection procedures utilized by the building industry. All of these industrial variations cannot be assimilated into a damage model of usable proportions. Hence, a significant amount of aggregating of somewhat dissimilar situations must be tolerated. The unavoidable outcome is additional uncertainty.

The difficulty of the damage modeling problem is further aggravated by an absence of data upon which to base any development. That data which does exist is not particularly reliable nor is it sufficiently detailed. It is unreliable because uniform procedures for assessing damage were not used in obtaining it. Available information is generally based on building owner reports on the cost of restoration. There is, however, a variety of repair options available to every owner. Thus, identical damage undoubtedly results in completely different repair costs. In addition, building owners are reluctant to divulge detailed information on damage repair for fear of adverse public reaction. Frequently, of course, detailed information does not even exist. Regardless of the source, however, data scatter is the end result.

Clearly the most expedient means of representing and modeling earthquake damage is via a probabilistic format. Several attempts at developing a statistical representation of earthquake damage have been made [26, 27, 28]. These procedures are very useful in making overall assessments of the damage potential for a particular geographical region or general structural type. They are, thus, helpful in site and structural type selection and for insurance assessments. As far as discerning moderately fine shades of difference between similar structures, however, they are not so helpful. For this purpose a deterministic approach is necessary wherein the cost of damage is modeled on the basis of specific structural response quantities rather than general structural properties and site conditions.

The difficulties in modeling damage costs noted above become especially acute when a deterministic format is adopted. Some accommodations are possible, however. For example, with regard to the earthquake loading, a detailed examination of the site geology and its fault system could produce likely locations, intensities, durations, etc. of any potential ground shaking and thus substantially reduce the uncertainty involved in this aspect of damage assessment. In addition, if local soil conditions are incorporated [13] in the design process a further reduction of uncertainty is achievable. As far as the variability in materials, design and construction practice is concerned some reduction seems possible via the development of a stable of damage models to accommodate the different possibilities. This requires the availability of a substantial body of detailed information on damage costs in actual structures. As noted above this information simply does not exist in the requisite quantity, quality or detail. Nevertheless, some effort has been directed towards this approach [29]. The developed damage models are understandably speculative in nature and largely unsubstantiated; they do appear to represent the best that is presently available, however.

Since a deterministic approach is used herein it is assumed that the earthquake loading is handled in a fashion somewhat similar to that given in the preceding paragraph with the building site composed of bedrock (or very firm soil). Rather than adopt the damage models of [29], however, simpler relationships which are more in keeping with existing data are utilized in the sequel.

The following discussion is subdivided according to the nature of the cost which is to be modeled, including structural and non-structural damage and down-time costs. In the final subsection the procedures involved in incorporating these models into an estimated LC due to earthquake overload are examined.

5.2.1 Structural Damage

One of the main difficulties encountered in modeling structural damage is defining it. Aside from [30] there appears to be little industrial guidance in this regard. Nor is there much indication on how structural damage has been assessed in the past. Logically there would seem to be two possible approaches to this important question. In the first approach structural damage would be defined as the amount of repair required for minimal restoration of a structure in order to insure its future adequacy. The second approach would be to define structural damage on the basis of a return of the structure to its original condition. What is actually done in reality, however, appears to be cosmetic in approach and a cross between the above two definitions of damage. Cosmetic repair is based on physical appearance (if it looks alright forget it) and location. Thus, if a damaged member is open to public display it is returned to its original condition, whereas if it is unseen it is repaired only if the assurance of structural adequacy requires it and then repaired only to the extent necessary. Note also that damaged structures in general have not been analyzed to determine which repairs are necessary or if contemplated repairs are adequate.

Clearly if present practice in structural damage repair is to be modeled, a very complex formulation is necessary. For steel framed buildings, however, structural damage is typically very light, to the point of being negligible, and probably does not warrant complex modeling. This conclusion is reinforced by the fact that supporting data is scant. Hence, for this effort simple modeling is embraced, based on the assumption that the structure is to be returned to its original condition.

The next aspect of damage modeling which requires attention is the selection of an appropriate measure for assessing structural damage. Inelastic energy absorption has been suggested as a possibility [29]. For wide flange members in moment resistant frames, however, the primary consequence of overloading is a loss in ductility. Hence, repair would involve restoration of the ductility of the structural members. The cost of damage would thus be proportional to the ductility demands made on the structure for a particular loading. Adopting this approach a simple damage model is developed in the following discussion.

Let μ be the ductility ratio for a member, which as noted in section 2.2 is given by

$$\mu = \begin{cases} \phi/\phi_p, & \text{for } \phi \geq \phi_p \\ 1 \quad, & \text{for } \phi < \phi_p \end{cases}$$

where ϕ is the member end rotation and ϕ_p is the rotation at the attainment of the plastic moment (see Fig. 5). Then the amount of ductility which must be restored is μ-1. If it is assumed that the cost of repair is directly proportional to the restored ductility, then this cost is represented as $c_o\ (\mu-1)$ where c_o is a constant of proportionality. Let p signify the cost of repair should the entire ductile capacity have to be restored, i.e., $p = c_o\ (\mu_c-1)$, where $\mu_c = \phi_u/\phi_p$. Define now the structural damage ratio, D_S, as the cost of repair divided by p, then

$$D_S = \frac{c_o\ (\mu-1)}{p} = \frac{c_o\ (\mu-1)}{c_o\ (\mu_c-1)} = \frac{\mu-1}{\mu_c-1}\ . \tag{5.6}$$

In terms of the damage ratio the cost of repair is given by $p\ D_S$. If it is assumed that there exist two potential plastic hinge points per member then the member damage cost is given by

$$\text{Member Damage Cost} = p\ D_S^{(1)} + p\ D_S^{(2)} \tag{5.7}$$

where the superscripts specify the associated hinge point. This relationship holds for both beams and columns and must be summed over all members to obtain a complete estimate of structural damage.

Loss of ductility is not the only type of damage which a structure can suffer. A significant amount of detailing damage also occurs [29]. As a first order approximation this type of damage can be assumed proportional to ductility loss and thus simply incorporated into p.

It should be noted that the above development corresponds roughly to the inelastic energy absorption approach. To see this, observe that the inelastic energy dissipated at a hinge point can be computed approximately by [31]

$$u = M_p \, (\phi - \phi_p) = M_p \, \phi_p \, (\mu - 1)$$

where u is the energy dissipated. Thus, the dissipative energy capacity is $u_c = M_p \, \phi_p \, (\mu_c - 1)$. If the cost of damage is assumed to be directly proportional to the inelastic energy dissipated then the damage ratio is computed as

$$D = \frac{c_1 \, M_p \, \phi_p \, (\mu - 1)}{c_1 \, M_p \, \phi_p \, (\mu_c - 1)} = \frac{\mu - 1}{\mu_c - 1}$$

where c_1 is a proportionality constant. This result is seen to be identical to that obtained using ductility loss as the damage measure. The ensuing development would thus be the same and result in a member damage cost as given above.

According to equations (5.6) and (5.7) the ductility demands and ductile capacity must be known in order to compute structural damage costs. As noted in section 2.3 the ductility demands can be computed from linear analysis using the ductility factor method. As far as the ductile capacity is concerned references [24, 31] contain excellent discussions of this important quantity.

5.2.2 Non-Structural Damage

Included in this category is damage to items such as interior and exterior walls, partitions, glazing, ceilings, plumbing, lighting fixtures, HVAC, stairs and elevator equipment. Taken collectively the cost of damage for these items is much more significant than structural damage in steel framed buildings. From the above list the principal contributions are from interior drywalls, glazing and masonry if present.

As was the case for structural damage there appears to be no consistent procedure presently available for the assessment of non-structural damage. Here again the cosmetic approach seems to be in vogue as far as owner assessment is concerned. This approach is, of course, quite acceptable for non-structural damage, albeit hard to model.

The selection of an appropriate measure for non-structural damage is simple. There is a clear consensus among investigators that story drift is the best indicator of this type of damage. Story drift represents the difference between the translational displacements of adjacent stories.

The simplest approach to modeling non-structural damage is to lump all the various contributions together and attempt to represent them collectively as a function of story drift. Not only is this the simplest approach it is also the only one for which sufficient data are available to aid in the development.

One attempt at such a development is given in [29]. The data used there are plotted in Fig. 16. As can be seen no clear correlation is present. Two things should be noted here: (i) the data are plotted on the basis of percent of construction cost rather than by damage ratio, and (ii) the collection of points located at the top of the figure represents only motels, whereas the lower group of data represents other types of structures. Presentation of this information in terms of damage ratio could thus result in a significant realignment. In order to readjust the data in this manner the percent total damage at each point must be divided by the percent of construction cost of the damaged items involved. The most significant contributions to the damage costs given in Fig. 16 are from drywall partitions and glass [32]. Hence, for each data point the percent of construction cost that these two items represent must be ascertained. Since this information is not available [29, 32] it must be determined indirectly.

To obtain approximate values for these quantities a survey of construction costs for recently completed buildings was conducted [33]. Only buildings of four stories or greater were included. The buildings were separated into two types: (i) buildings for which a high interior wall density would normally be expected, e.g., hotels, motels, hospitals, apartments, etc., and (ii) buildings in which a moderate interior wall density would be most likely, which in this survey ended up being composed almost completely of office buildings. Low interior wall density structures such as retail stores, manufacturing plants and industrial buildings were not included in the compilation. The results of the survey indicate that for the high wall density buildings the combined cost of drywalls and glass amounts to 10.24% of the total construction cost whereas in the office buildings this value is 6.29%.

Using the above values along with the data of Fig. 16, the revised non-structural damage chart of Fig. 17 is obtained. The damage ratio here is the cost of damage divided by the cost of construction of the items in question. As can be seen a fairly clear correlation emerges. A least squares fit which is forced through the origin results in the solid line given in Fig. 17.

Using the relationship given in Fig. 17 to compute the non-structural damage ratio, D_N, the cost of damage per story can be developed. Let f represent the value of glass and drywall partitions for a particular floor and d_f represent the estimated maximum non-structural damage costs for the same floor, then $d_f = f\ D_N$. The complete estimate of non-structural damage is obtained by adding together all the floor values.

5.2.3 Down-Time Costs

Damage repairs frequently require closing off sizable portions of a building to normal use. The functions performed in these parts of the building are either relocated or shut down temporarily. In either case what results is a loss of revenue, referred to here as down-time or inconvenience costs. Modeling this type of cost is made difficult by the fact that, as in the case of non-structural damage, what is done in this regard is highly dependent on the building owner and what he is willing to do. Again little data are available to aid in model development. This being the case, simple modeling is all that is warranted. As a first order approximation it is assumed here that revenue losses are directly proportional to damage costs.

To get a rough idea of what range of inconvenience costs can result, the data of [34] were examined in terms of the ratio of inconvenience cost to total damage cost. This examination revealed considerable scatter with reported down-time costs ranging from 0% to 300% of the total damage costs. The main body of data, however, varied only between 0% and 30% with a preponderance of 0% data points. Obviously in order to make an estimate of this type of cost some assessment of the susceptibility of a building to inconvenience costs must be made along with a review of the previous practices of the building owner.

5.2.4 Lifetime Cost Estimate

The damage cost models developed in the preceding subsections are deterministic in format and apply only to individual earthquakes. Hence, to develop a lifetime cost an expected earthquake profile must be identified, the damage cost for each earthquake in the profile computed and the resultant costs summed over all expected earthquakes. The next item of business then is the establishment of an expected earthquake profile for the particular site and planned service life of the building being designed.

The frequency of occurrence of earthquakes is reasonably well described by an equation of the form [35]

$$n = \frac{A\ N_o}{B}\ e^{-M/B} \tag{5.8}$$

where ndM is the number of shocks with magnitudes between M and M + dM in area A, N_o is a measure of the average seismicity of a region and represents the annual number of shocks per unit area, B is a distribution parameter describing seismic severity and M is the Richter magnitude. For the highly seismic region of southern California these parameters assume the values $N_o = 1.7/mi^2$ and $B = 0.48$ based on the 29-year period, 1934-1963. It is generally assumed that the number of earthquakes drops off from that predicted by equation (5.8) at the higher magnitudes and eventually goes to zero. For southern California the zero value, i.e., the upper bound, is typically taken as $M = 8.5$, [35].

To develop a similar equation in terms of ground acceleration, assume an earthquake intensity attenuation profile as given in Fig. 18. Using this profile affected area curves in terms of ground acceleration can be developed and are given in Fig. 19 [36, 37]. From the curves of Fig. 19 a table of covered area versus ground acceleration is developed as shown in Fig. 20. The values in this table represent the estimated amount of area over which the specified ranges of ground acceleration will exist during earthquakes of the given magnitude. Assuming a fixed fault direction, substitution of these areas into equation (5.8) yields an estimate of the number of ground motions which can be expected within the specified range of ground acceleration due to earthquakes of the given Richter magnitude. A table of such numbers based on the southern California values for N_o and B is given in Fig. 21. Note that the table is given in terms of gravitational acceleration, g. Thus, the mean values in this table represent the expected number of earthquakes per one hundreth of g, times one hundred. A least squares curve fit to these mean values results in

$$n = 3.44\ e^{-15.25a} \tag{5.9}$$

where a is the ground acceleration divided by g and nda is the number of earthquakes with ground acceleration between a and a + da. These values are per annum and thus must be multiplied by the structure's life expectancy in order to obtain a life estimate. The curve of equation (5.9) is assumed to drop off at the higher ground accelerations in a manner similar to that assumed for equation (5.8). The exact shape of the drop-off is not important; it is sufficient to recognize that the drop-off is sharp and goes to zero at $a = 0.5$. Equation (5.9) in conjunction with a service life estimate represents the required earthquake profile.

To obtain an estimate of the damage costs for a building the expected damage versus ground acceleration must first be computed using the damage models developed in the previous subsections. These quantities are then multiplied by the expected number of earthquakes at each value of ground acceleration to obtain a curve which represents the lifetime damage costs versus ground acceleration. The area under this curve yields the estimated total lifetime damage costs for a particular building.

As an example of such a computation consider the one story optimal frame of [38]. This frame is 150 inches tall and 300 inches wide with beam and column moments of inertia of 223 in^4 and 235 in^4 respectively. The beams carries 40 kips of distributed dead/live load. From [39] the lateral stiffness of this frame is 28200 kips per inch with a natural frequency of 2.62 Hertz. Assuming 5% of critical damping in the Newmark-Hall procedure results in a story drift of $\delta = 3.7a$ where δ is in inches. Using this in the non-structural damage model of section 5.2.2 with an assumed glass and drywall cost of 10% of the construction cost and employing equation (5.9) with a 50-year service life yields an expected lifetime damage profile of

$$d_t = 4540\ a\ e^{-15.25a} \tag{5.10}$$

where d_t is the total expected damage versus ground acceleration. Structural damage costs can also be computed but for this case were found to be negligible. A plot of equation (5.10) is presented in Fig. 22. As can be seen most of the structural damage results from ground accelerations of less than 25% g with the peak in the curve occurring at 6.56% g. The area under this curve is easily computed from equation (5.10), in the general case, however, numerical integration is necessary. Using the trapezoidal rule exact integration points for the curve of Fig. 22 are a_i = 0, 0.065, 0.294, 0.5. While these values are exact only for the one story structure discussed here they should yield reasonably good results for multistory buildings as well since, roughly speaking, a multistory frame is simply several one story frames placed on top of one another. More will be said on this later.

This completes the specification of the decision motivator. The motivator is composed of the expected damage costs for a building, as represented in Fig. 22, plus the construction cost as developed in section 5.1. This motivator is a function only of the descriptive variables for the option sets, i.e., the design vector. It remains now to develop an option search mechanism. To guide the development of such a mechanism a close examination of the problem at hand is appropriate. This is the subject of the next two sections.

6. SYSTEM CONSTRAINTS

In order to conduct a detailed examination of the design problem at hand an explicit formulation of the system constraints discussed in section 2.2 must be presented. For the purpose of this exposition it is not necessary that these constraints be formulated exactly as they appear in standard practice but rather that they represent only good approximations. To this end all of the internal force constraints are constructed in terms of moments rather than stresses as typically done in practice. Thus, internal force limits are based on member plastic moment rather than yield stress. In the case of beam members the constraint functions which result from the moment and stress approaches are virtually identical, as will be shown; differences occur only when dealing with columns. The important advantage of using the moment approach is that it facilitates construction of the constraints on a unified basis resulting in a reduction in handling complexity. It should be noted that this approach is not without precedent and corresponds to the ultimate-strength design procedure used for reinforced-concrete frames [20].

The discussion which follows is broken into two subsections, the first dealing with static load constraints and the second with dynamic load constraints. In both sections an attempt is made to prioritize constraints according to their importance in the design process. Those design limitations, which could possibly play a major role in design selection, are referred to as primary constraints and those which are not expected to participate are labeled as secondary constraints. The intent here is to accomodate only primary constraints in the actual design process with a check of secondary constraints made only upon final design selection. This approach appears to have promising potential in making a significant reduction in the cost of completing a design selection effort [38].

6.1 Static Loading Constraints

Static loads are placed on the structure in order to simulate normal operating conditions. The design limitations imposed under these circumstances are to insure adequate performance of the system in its day-to-day use. The essential limitations in this regard are member force restrictions, beam deflection constraints and sidesway stability requirements. Each of these items is discussed in turn in the following.

Internal member force constraints are imposed to insure that yield stress limits are not exceeded, so that permanent distortion of the building frame is avoided. These constraints are thus imposed on the maximum stresses or moments within each member.

For beam members there are three possible locations where the maximum moment could occur, at either end of the beam or in the middle. Since the structures considered here are symmetric and the loading is symmetric the beam end values are identical, hence, only one end, the left end herein and the middle need be monitored.

Recalling from section 2.3 that the static analysis is to be done in two parts, the moment constraint for the left end of the beams is

$$|M - w\,L^2/12| \leq c\,M_p$$

where M is the left end-moment from the matrix structural analysis, $-\,wL^2/12$ is the left end-moment from the fixed-end analysis, w is the distributed loading, L is the beam length, and c is a reduction coefficient. The absolute value in this expression produces a discontinuous derivative which frequently leads to difficulties in many automated design schemes. In order to eliminate the absolute value two constraints are adopted in place of the one above:

$$M - w\,L^2/12 \leq c\,M_p \tag{6.1}$$

$$-\,M + w\,L^2/12 \leq c\,M_p \quad . \tag{6.2}$$

For the middle of the beams the moment constraint is

$$|M + w\,L^2/24| \leq c\,M_p$$

or

$$M + w\,L^2/24 \;\leq c\,M_p \tag{6.3}$$

and

$$-\,M - w\,L^2/24 \;\leq c\,M_p \tag{6.4}$$

where again M is the matrix moment at the left end and $wL^2/24$ the value of the moment at the middle of the beam corresponding to the fixed-end solution.

With regard to the order of these constraints it is expected that $0 < M < wL^2/12$ where the bounds on M represent rigid joint and pinned joint conditions respectively. If this is the case then (6.2) and (6.3) represent primary constraints and (6.1) and (6.4) secondary. Because of earthquake requirements it is further expected that $M < wL^2/48$ (ie., nearly fully rigid joints). Under this condition (6.2) is the only primary constraint with (6.3) joining the ranks of secondary constraints.

In order to develop a feel for what the reduction coefficient c should be, let M_T represent the sum of the matrix and fixed-end solutions, then the above constraints can be represented in the form $M_T \leq cM_p$. Now recall from section 3 that Z = 1.13S, so that

$$M_T \leq c\,M_p = 1.13\,c\,\sigma_y\,S,$$

where σ_y is the yield stress. Thus

$$M_T / S \le 1.13 \; c \; \sigma_y \quad , \tag{6.5}$$

which is the usual yield stress condition. For properly braced compact sections AISC allows $0.66 \; \sigma_y$ as the upper bound in (6.5), hence $1.13 \; c = 0.66$ or $c \cong 0.6$.

For columns there are two potential sites for occurrence of maximum moment, at either the top or bottom of the column. Since the structure and loading are symmetric, only one column per story need be monitored. At the top of the column the moment constraint is $|M| \le cM_p$ or

$$M \le c \; M_p \tag{6.6}$$

and

$$- M \le c \; M_p \tag{6.7}$$

where M is the top-moment. An identical set of limitations prevail at the bottom end of the column, where M is the bottom end-moment.

As far as order is concerned both the top and bottom end-moments are expected to be positive under the standard matrix displacement method of moment labeling (see [38]). Hence, only the two constraints resulting from (6.6) are primary with the two constraints from (6.7) secondary.

For columns the plastic moment must be modified to reflect any axial loading. AISC suggests

$$M_p = \begin{cases} \sigma_y \; Z & , \text{ for } P/P_y \le 0.15 \\ 1.18 \; \sigma_y \; Z \; (\; 1 - P/P_y \;) & , \text{ for } P/P_y > 0.15 \end{cases} \tag{6.8}$$

where P is the axial load and $P_y = \sigma_y A$ is the yield load. Because of this modification the moment constraints (6.6) and (6.7) do not coincide with the usual stress constraints. They are close, however.

To insure that human discomfort does not result through "soft-floors" or that damage does not occur because of excessive sagging, limitations are generally placed on the vertical displacement of the center of beams. AISC recommends an allowable displacement of 1/360 of the beam length for live loading. The displacement constraint is thus given by

$$\frac{L^2}{EI} \left(\frac{wL^2}{384} + \frac{M}{8} \right) \leq \frac{L}{360} \tag{6.9}$$

where w is the distributed live load and M is the matrix solution for this load at the left end of the beam. For typical values of the various parameters it seems very likely that this constraint is secondary to (6.3).

The final item to consider under static loading is the lateral stability of each of the stories, commonly referred to as sidesway stability. It has been shown that sidesway stability can be treated with reasonable accuracy using standard column buckling equations [20]. Hence, sidesway constraints can be expressed as

$$P \leq c\, P_{cr} \tag{6.10}$$

where P_{cr} is the column buckling load. This load is usually computed from the Column Research Council formula:

$$P_{cr} = \begin{cases} \dfrac{\pi^2\, E\, A}{(\, KL/r\,)^2} & , \text{ for } KL/r \geq 128.25 \\[2ex] P_y \left[1 - \dfrac{\sigma_y}{4\, \pi^2\, E} \left(\dfrac{K\, L}{r} \right)^2 \right], & \text{for } KL/r < 128.25 \end{cases}$$

where K is an effective length coefficient. The effective length coefficient is normally found via a nomographic representation of the solution of the equation

$$\frac{G_A\, G_B\, (\, \pi/K\,)^2 - 36}{6\, (\, G_A + G_B\,)} = \frac{\pi/K}{\tan(\pi/K)} \ . \tag{6.11}$$

The subscripts A and B in this equation refer to the joints at the two ends of the column being considered. For fully rigid joints G is defined as

$$G = \frac{\Sigma\ (I/L)_C}{\Sigma\ (I/L)_B} \tag{6.12}$$

where the subscript C signifies summation over all the columns entering a joint and the subscript B indicates summation over all the beams entering the joint. For joints with semirigid beam-column connections an effective beam stiffness must be used in (6.12), it is given by [19]

$$\left(\frac{I}{L} \right)_{B_{eff}} = \frac{I}{L} \left[\frac{3}{4(L_A /L) - (L/L_A)} \right]$$

where

$$L_A = L + 3 (EI/k)$$

and k is the connection stiffness.

Computer implementation of equation (6.11) is probably best accomplished via a root finding procedure. For example use of Newton's method on the equation

$$f(\pi/K) = \frac{G_A\ G_B\ (\pi/K)^2 - 36}{6\ (G_A + G_B)} - \frac{\pi/K}{\tan(\pi/K)} = 0$$

where

$$f'(\pi/K) = \frac{G_A\ G_B\ (\pi/K)}{3\ (G_A + G_B)} + \frac{(\pi/K)\ \sec^2(\pi/K) - \tan(\pi/K)}{\tan^2(\pi/K)}$$

results in the iterative relationship

$$(\pi/K)_{i+1} = (\pi/K)_i - f(\pi/K)_i \ / \ f'(\pi/K)_i \quad .$$

In most practical situations neither G_A nor G_B will be less than one, in which case $(\pi/K)_0 = 2.42$ makes an excellant initial point.

To establish a value for c in equation (6.10) note that AISC suggests a factor of safety of approximately 1.67 for very short columns, corresponding to a c of 0.6, whereas for slender columns AISC calls for a factor of safety of 1.92, which gives c = 0.52.

Because of the lateral strength requirements of earthquake resistant frames it is strongly expected that the above sidesway stability requirements constitute secondary constraints [38] and will thus not play a prominent role in the design process.

6.2 Dynamic Loading Constraints

Recall from section 2.2 that the dynamic characteristics of a structure are to be constrained on the basis of dual design criteria. These criteria are stated as:

(i) the structure should respond elastically to a moderate earthquake of an intensity reasonably anticipated within its lifetime

(ii) during a maximum credible earthquake the structure may yield significantly but must avoid collapse.

The first item of business then is the selection of two ground motions, referred to herein as design earthquakes, which are representative of the above conditions. Design earthquakes are typically chosen on the basis of their probability of occurrence [11, 13]. A sample probability of occurrence curve is shown in Figure 23. This curve was generated on the basis of a 50 year life expectancy for a southern California site using the affected area curves of Figure 19. The procedure for constructing such curves is outlined in [36]. The plot in Figure 23 represents the probability that an earthquake of given ground acceleration or greater will occur at least once during a 50 year period. Moderate earthquakes (criterion (i)) are typically selected on the basis of a 50-80% probability of occurrence whereas strong earthquakes (criterion (ii)) are picked to have a 5-10% probability of occurrence, both for a life expectancy of roughly 50-70 years.

Use of the above procedure results in the selection of two peak ground acceleration values reflecting moderate and strong earthquakes. As noted in section 2.3 specification of peak ground acceleration is sufficient to completely characterize a particular ground motion for the purposes of linear elastic analysis. Selection of a structural damping value facilitates the construction of structural response spectra for use in a mode superposition approach to dynamic analysis. Hence, the peak ground acceleration values chosen via the probability of occurrence curves represent the required design earthquakes.

Normally in the specification of dynamic system constraints dead/ live load effects on the beams are accomodated in addition to the earthquake loading results. This is the format followed here. Since the analysis is done in three parts, namely a dynamic analysis and a two-part static analysis, the constraints are written in similar fashion. Frequently a reduction of the live load from that specified for the static operating constraints is allowed in the earthquake analysis. Such will not be the case here. The dead/live load stipulated for the operating constraints is employed for the dynamic constraints as well.

For a moderate earthquake the structure is to respond elastically, hence, the maximum member moments throughout must be less than each corresponding member yield moment, M_y. For a beam there are three potential locations where the maximum moment could occur: at either end of the beam or some point in the middle [38]. Since the dynamic loading is antisymmetric and the structure symmetric the dynamic moments are equal in magnitude but opposite in application on the two sides of the structure. Recall now from section 6.1 that the static moments are equal on the two sides of the structure. From these results it is easily shown that the moment constraints for the two ends of the beams are redundant. Hence, only the left end is monitored here. One additional point should be mentioned. Because the rss procedure is used in generating the dynamic moments these moments are positive everywhere in the analytical solution. In reality negative moments are equally likely however, hence, both possibilities must be accounted for. With this in mind the left end beam constraints for the moderate earthquake are

$$|M_s + M_d - wL^2/12| \leq M_y = c\ M_p \tag{6.13}$$

and

$$|M_s - M_d - wL^2/12| \leq c\ M_p \tag{6.14}$$

or

$$M_s + M_d - wL^2/12 \leq c\ M_p \tag{6.15}$$

$$- M_s - M_d + wL^2/12 \leq c\ M_p \tag{6.16}$$

$$M_s - M_d - wL^2/12 \leq c\ M_p \tag{6.17}$$

$$- M_s + M_d + wL^2/12 \leq c\ M_p \tag{6.18}$$

where M_y has been written in terms of M_p as shown in (6.13). The subscript s signifies the static moment matrix solution and the subscript d the dynamic moment. The possibility of a positive or negative dynamic moment is handled by specifying the two sets of constraints (6.13) and (6.14). The first item to note with regard to constraints (6.15) - (6.18) is that because $M_d \geq 0$, (6.18) supersedes (6.16) and (6.15) supersedes (6.17), so that of the four constraints only (6.15) and (6.18) are necessary. In addition, as noted in section 6.1, since $0 \leq M_s \leq wL^2/12$ it can be concluded that (6.18) is a primary constraint and (6.15) is secondary.

The mid-beam constraint for the moderate earthquake is [38]

$$\left| (wL^2/24) + M_s + (2M_d^2 /wL^2) \right| \leq c\ M_p$$

or

$$(wL^2/24) + M_s + (2M_d^2 /wL^2) \leq c\ M_p \qquad (6.19)$$

and

$$- (wL^2/24) - M_s - (2M_d^2 /wL^2) \leq c\ M_p \qquad (6.20)$$

where the given moments are from the left end of the beam. Quite clearly $-M_d$ need not be considered here. Since it is expected that $M_s \geq 0$, (6.19) is a primary constraint and (6.20) is a secondary constraint. Indeed, it would appear that (6.20) may not need to be considered at all.

As was shown in [38], (6.19) can be combined with (6.15) to form the single constraint

$$c\ M_p \geq \begin{cases} M_d + M_s - (wL^2/12) & \text{for } wL^2/4 \leq M_d \\ M_s + (2M_d^2 /wL^2) + (wL^2/24) & \text{for } wL^2/4 > M_d \end{cases} \qquad (6.21)$$

Essentially then, only two constraints are needed for each beam.

The maximum moments in the columns occur at the top end and the bottom end. Because of the symmetries mentioned previously only one column per story need be considered. For either end of the column the moment constraint is

$$|M_d + M_s| \leq c\ M_p$$

and

$$|- M_d + M_s| \leq c\ M_p$$

or

$$M_d + M_s \leq c\ M_p \qquad (6.22)$$

$$- M_d - M_s \leq c\ M_p \qquad (6.23)$$

$$- M_d + M_s \leq c\ M_p \qquad (6.24)$$

$$M_d - M_s \leq c\ M_p \qquad (6.25)$$

where the moments come from the appropriate column end. Since $M_d \geq 0$, (6.22) supersedes (6.24) and (6.25) supersedes (6.23). It is expected that $M_s \geq 0$ hence (6.22) is primary and (6.25) is secondary. Thus, there are two primary and two secondary constraints per column.

An appropriate value for the constant c in all of the above moderate earthquake constraints can be found by noting from equation (6.13) that $c = 1/f$ where f is the member shape factor. For wide flange sections the largest shape factor is [20] 1.18, hence, a good value for c is 0.85 .

For the strong earthquake design requirements strong column-weak girder provisions are invoked. According to the strong column-weak girder philosophy, inelastic activity should be confined to the beams as much as possible. In terms of ductility ratio this means that the ductility demands of each member must be less than some specified allowable, which for the columns is one or close to one. For this discussion let M_T represent the total maximum moment (i.e., the sum of the static and dynamic moments) in a particular member. Then by the ductility factor method discussed in 2.3 the general form of the major earthquake constraints is $M_T / M_p \leq \mu_a$ where μ_a is the allowable ductility. This constraint can be stated alternatively as $M_T \leq \mu_a M_p$. As can be seen this equation is identical in form to (6.13) - (6.25) with $c = \mu_a$. Hence, all of the constraint developments for the moderate earthquake apply to the strong earthquake with c equal to the allowable ductility in each member.

With reference to ordering it is clear that the similarity in the form of the moderate and strong earthquake constraints will facilitate categorization of these limitations. It does not appear however that such an ordering is possible a priori. Instead, at least one design analysis is necessary to assist in establishing such order. Ranking the moderate and strong earthquake constraints could be made part of the initial design assessment.

7. DESIGN SPACE EXAMINATION

The point has now been reached where an indepth examination of the design problem outlined in the preceding sections is possible. This examination will be conducted in the space whose elements are the components of the design vector, referred to as the design space. The intent of this examination is to develop an intuitive feeling for what the design space looks like in terms of its mathematical description. It is hoped that a qualitative assessment of the design problem will provide some insight as to how to best approach the formulation of an option search procedure.

To facilitate the investigation a four-story structure, illustrated in Fig. 24, is selected for detailed analysis. As indicated in the figure, it is assumed that the frame is repeated at 24 foot intervals into the plane of the illustration. The particular frame being examined is the end-frame. For the roof a 100 lb/ft^2 dead load and a 25 lb/ft^2 live load is applied, resulting in a 45 kip distributed loading on the roof beam. The floors sustain a 125 lb/ft^2 dead load and a 75 lb/ft^2 live load which yields a 72 kip distributed load on each floor beam. The moderate design earthquake is chosen to have an 80% probability of occurrence, which Fig. 23 shows to be a 0.12 g peak ground acceleration. The strong earthquake is selected on the basis of a 5% probability of occurrence, which through Fig. 23 equates to a 0.35 g peak ground acceleration. The reduction coefficient for the static loading moment constraints (see equations (6.1) - (6.7)) is 0.6 and for the dynamic loading (moderate earthquake) moment constraints (see equations (6.13) - (6.25)) is 0.85. The column buckling reduction coefficient (see equation (6.10)) is 0.6. A deflection of approximately one inch is allowed at the center of each beam under live load. Ductility allowables are 6 for the beams, 2 for the first- and fourth-story columns and 1 for the second- and third-story columns. In an attempt to maintain some consistency among prices most construction cost rates are taken from [7]. The cost of A36 steel is \$13/cwt, A514 steel \$26/cwt, and the transportation of steel \$0.21/cwt. The cost of welding is \$449.80/cwt of weld metal deposited and the cost of painting \$0.10/ft^2. The rate of job overhead and profit are each assumed to be 10% of the construction cost. For the purpose of a damage cost estimate the top floor is assumed to have a value of \$310.82 for the glass and drywall partitions supported by the end-frame. The other floors are each assumed to have a value of \$233.11 (see appendix for how floor values are estimated). The down-time cost is computed at 10% of the total damage cost. Structural damage is not accounted for.

The arrangement of the components of the design vector X is given in Fig. 24. The moments of inertia of the members are listed first, followed by the connection stiffnesses with the member plastic section moduli listed last (in parenthesis).

Clearly, direct graphical illustration of the design space is not possible due to the large number of components in the design vector. Hence, a series of two-dimensional design problems is examined instead. These problems are arranged by selecting two components at a time from the design vector and varying them relative to each other while all other components in the design vector remain fixed. It is hoped that with a judicious selection of component pairs a reasonably accurate description of the design space can be presented.

There are numerous pairs of components of interest. In the following discussion each pair is taken up in turn and examined in terms of what is revealed about the design problem in general. The basic design vector about which the various design variations will revolve is given by

$$X = (655,\ 645,\ 2322,\ 627,\ 1893,\ 709,\ 1944,\ 1020,\ 9(10^9),\ 9(10^9),\ 9(10^9),\ 9(10^9),\ 87,\ 102,\ 213,\ 99,\ 184,\ 112,\ 188,\ 158)^T. \tag{7.1}$$

This vector represents the optimal (i.e., minimum LC)design for the structure of Fig. 24 with system parameters as given above. It represents an optimal design only with the connection stiffnesses fixed as given in (7.1). The reason for this will be explained subsequently.

As mentioned above, the design spaces in the sequel are generated by varying the members of a component pair in conjunction with one another while all other components of the design vector remain fixed. This format will be modified only in the case where one of the components of a pair is a moment of inertia. Then, in order to keep the design space within the structural member option set, the associated plastic section modulus, if it is not the other component of the pair, will be varied in accordance with the lower bounds in equations (3.3) and (3.4).

The reason design vector (7.1) is optimal only for fixed connection stiffnesses is best seen by examining the moment of inertia versus connection stiffness for a representative beam and column. The first variable pair isolated for examination then is the fourth-story beam moment of inertia and the fourth-story beam-column connection stiffness, i.e., X_1 and X_9. This two-dimensional design space is shown in Fig. 25. The hatched lines denote system constraints with the unhatched side of the curves representing usable designs and the hatched side unusable designs. Constraint number 2 corresponds to equation (6.2) for the fourth-story beam, 3 to equation (6.3) and 42 with the moderate earthquake constraint (6.18). As can be seen in the illustration 42 parallels 2 and thus never participates in the design process. It is clearly a secondary constraint. The cost lines in the figure portray the decision motivator in terms of the construction cost of the member (or members) being examined as follows:

$$\text{Cost} = \frac{100\ [LC(X) - LC(X^{o})]}{\text{Member Construction Cost at Optimal}}$$

where LC(X) is the lifetime cost as a function of the present design vector X and X^{o} is the starting design vector (7.1).

As can be seen in the figure the decision motivator (cost) is unbounded with regards to the connection stiffness so that an infinitely stiff (i.e., fully rigid) connection is called for. Hence, $k = 9(10^{9})$ is chosen as being close enough for the purpose of this study. The implication of this result is that semirigid connections have no place in lateral force resistant frames where the lateral force is provided by earthquakes in highly seismically active regions. Note, however, that if earthquake loading is not present then the motivator is essentially one of least weight with cost lines roughly parallel to the X_9 axis. In this case the optimal design occurs at the junction of constraint 2 and 3. This solution represents a semirigid connection with an 86% rigidity. Hence, there clearly exists some range of earthquakes, possibly very small, for which semirigid connections could prove useful. What values of seismicity would prevail among these earthquakes is the next logical question, but its answer is beyond the scope of this investigation.

To develop further support for the above conclusions the second story moment of inertia X_6 versus the second story connection stiffness X_{11} is now examined. This design space is shown in Fig. 26. The system constraints shown are for the second story column with 57 and 59 corresponding to moderate earthquake inequality (6.22) for the top and bottom moments of the column. Constraints 81 through 84 correspond to the strong earthquake inequalities (6.22) and (6.25) for both the top and bottom ends of the column. Constraints 81 and 83 are clearly primary with the remainder secondary. Again it can be seen that the decision motivator is unbounded in the connection stiffness. A similar examination of all the connections of the structure of Fig. 24 reveals an identical situation to exist at each. At this point it is clear that the conclusions of the preceding paragraph are indeed correct and that only fully rigid connections should be considered in conjunction with the class of problems being explored here.

Note in passing that the optimal design of Fig. 25 is unconstrained whereas in Fig. 26 it is partially constrained. In the design space of Fig. 26 the optimal lies on the surface of constraint 81 which is the strong earthquake limitation on ductility demand. More will be said about this later.

In order to determine if suboptimization as discussed in section 3 is feasible the moment of inertia and plastic section modulus of a representative column and beam should be studied. Thus, the next design variable pair isolated for examination is the second story column moment of inertia X_6 and plastic section modulus X_{18}. Fig. 27 illustrates the resultant design space. The unlabeled constraints in this figure are the option set limits for column sections.

From the illustration it is clear that the optimal design for this two-dimensional problem is located at the juncture between system constraint 81 and the option set lower bound. Even if 81 were not present the optimal design would still lie on the lower option set bound. Recall that this lower bound corresponds to the economy sections as described in section 3 and represents the reduced option set which results from suboptimization. Since the optimal solution for the general problem lies within the suboptimized option set it would appear that suboptimization does offer a viable approach to simplifying the class of design problems addressed here.

To further explore this possibility consider the design space formed by the fourth story beam moment of inertia and the associated plastic section modulus. This design space is depicted in Fig. 28. As before the unlabeled constraints represent the option set bounds, this time for beams. Here again the optimal design lies on the lower option set bound, i.e., within the set of economy sections. A check of all the other members shows that a similar result exists in each. It is clear from Figs. 27 and 28 that the optimal will always lie on the lower option set bound regardless of whether the depicted system constraint functions intervene or not. Hence, suboptimization does work for the type of problems examined here and should be used to reduce the design problem complexity. Caution should be used in extending this conclusion to other situations, however, since it is basically only applicable in the face of the moment-type constraints which dominate the problems herein. Should the buckling constraints become prominent or beam depth restrictions be imposed, this conclusion would not hold in all cases.

All of the cost lines in Figs. 27 and 28 which extend beyond the option set bounds represent extrapolations since the cost models were developed on the basis of information for points only within these bounds. As might be expected some interesting results occur in extrapolation. For example, the cost goes to plus infinity and minus infinity on either side of a curve running roughly parallel to the option sets in the upper left-hand portion of both figures (not depicted). This strange result occurs as a consequence of the cost of painting model which is based in part on equation (3.5) and is the cause of the difficulty. The interesting cost line in the lower right hand portion of Fig. 28 (Cost = -109) is also a result of the cost of painting model. It is very apparent that extrapolation is not a good idea and conclusions based on the extrapolated cost lines of Figs. 27 and 28 should not be made.

In [38] it is shown that an important consideration in developing an option search procedure is member interdependence. For minimum weight structures such as those examined in [38] member interdependence manifests itself through the constraint functions. When employing a lifetime cost decision motivator, however, the motivator itself also enters the picture.

To examine member interdependence consider the two-dimensional design space formed by the third-story column moment of inertia X_4 and the second-story beam moment of inertia X_5. The prominent constraint functions in this design space are shown in Fig. 29. Constraints 10 and 11 correspond to inequalities (6.2) and (6.3); 45, 46, and 53 to moderate earthquake limits (6.20), (6.18), and (6.22) for the top end of the column and constraints 77 and 79 to strong earthquake restriction (6.22) for the top and bottom end moments. The important point to observe in this figure is the general lack of member interdependence, i.e., the constraint curves are roughly parallel to their respective member axes. This is in direct contrast to the observations made in [38] where considerable member interdependence was found. This conflict is clearly attributable to the fact that in [38] relatively flexible minimum weight structures were being dealt with while herein a fairly rigid structure is being examined. Thus, it appears that member interdependence recedes as frame rigidity increases. This observation is certainly born out by the appearance of the constraint curves in Figs. 25 and 26.

Isoplots of the decision motivator for the preceding design variable pair are given in Fig. 30. The major feature of these plots is that the principal directions in the cost surface (eigenvectors of the Hessian of the cost function) appear to be parallel to the axes. Hence, very little, if any, member interdependence is manifested via the decision motivator.

To further pursue this theme the moment of inertia for the fourth story column X_2, verus the moment of inertia for the third story column X_4, is examined. This comparison is plotted in Fig. 31. Again it is clear that little member interdependence is introduced via the system constraints. Also, as before, the principal directions of the cost surface appear to be parallel to the axes. Note that this result is also present in Figs. 25 and 26. It is quite apparent then that very little member interdependence exists at the optimal solution for the four story problem of Fig. 24. The principal implication of this conclusion is that the sizing of the various members can take place nearly independently of one another, i.e., the member sizing decisions are uncoupled. This has major ramifications in selection of an option search procedure as will be seen in the following section.

Several important results have been developed in this section. First, fully rigid connections are desirable for structures of the class considered here. Second, suboptimization is a viable approach to reducing the number of required design variables. Third, member sizing decisions are nearly uncoupled. One additional observation should also be made. Note that the optimal design vector given by (7.1) represents only a partially constrained solution. That is, only one constraint, 81, is active for 8 design variables (where the plastic section moduli are regarded as dependent variables and the connection stiffnesses are fixed). Note further that the completely unconstrained optimal design lies only slightly below the constraint surface. This can be clearly seen in Fig. 26. These facts would suggest that the constrained and unconstrained optimals may in general lie quite close to one another within the design space. If true, knowledge of the location of one optimal could be used to quickly ascertain the location of the other. This is an interesting idea which could be put to good use if one of the optimals were easier to find.

As a result of the above conclusions. two refinements are imposed in the sequel. First, only fully rigid frames are considered. Second, member option sets are restricted to include only economy sections. Thus, the design problem originally addressed in section 1 is reduced to one of member sizing using the moments of inertia as the design variables. The other results of this section are effectively utilized in establishing the methodology developed in the following section.

8. OPTION SEARCH ALGORITHM

The primary purpose of the algorithm to be developed in this section is to complete the member sizing operation in accordance with the restrictions and objectives of the design process as presented in the preceding sections. In its most general setting this design operation would start with the formulation of an initial design and conclude with the specification of the best or optimal design. Within this general setting there are two secondary operations which are frequently of use by themselves in actual design development. In the first an initial design is available but is not usable, that is it does not satisfy all the design requirements. What is necessary in this case is a procedure for developing a usable design starting from the initial design. The second useful suboperation assumes an initial design which is usable, but is weak in terms of material utilization. In this situation a design improvement procedure is required.

In order to accomplish the objective of the general design problem as will as provide apparatus to conduct the two suboperations a three phase algorithm seems most appropriate. In the first phase an initial design is formulated. The second phase develops usable designs from unusable designs and the third phase provides for design improvement. An algorithm based on this three phase approach is given in the appendix. The techniques used in accomplishing each phase are discussed in turn in the sequel.

The procedures used for developing an initial design depend primarily upon the purpose for which the initial design is formulated. Insofar as serving as the initial point in a search for an optimal is concerned it would seem that the initial design is best typified as a good, inexpensive estimate of the location of the optimal. The unconstrained optimal presented in the previous section would seem to be a good candidate for an initial design. As noted, it appears to be in close proximity to the constrained optimal. In addition, because the sizing decisions are uncoupled it should prove to be inexpensive to find. Hence, it is adopted for this purpose.

The problem of formulating an initial design has thus been reduced to that of finding an unconstrained optimal design. In problems where the decision variables are uncoupled from one another, coordinate descent algorithms can prove to be very effective [40, 42] in this endeavor. Thus, a variant of such procedures is adopted here.

Let f represent the cost function, then the essence of the coordinate descent approach is contained in the expression

$$\min_{x_i} f(x) \quad \text{for all } i \ . \tag{8.1}$$

As this expression indicates the unconstrained optimal is sought by searching in turn in each coordinate direction x_i . If there are n components in the design vector, then for an uncoupled cost function, n line searches are required to find the optimal.

An efficient method for conducting the above line searches is via curve fitting. For example if the value of the cost function and its derivative are available at two points along a line then a cubic equation can be fitted through these points and the minimum of the cubic taken as an estimate of the solution to equation (8.1). At the very least, this procedure requires, in addition to the initial design analysis, one new analysis for each line search. Hence, to find the optimal, n + 1 analyses are required. For large structural systems this represents an enormous number of expensive analyses and is clearly unacceptable.

In order to improve upon this situation a simultaneous coordinate search procedure is adopted instead. Using this approach searches in all coordinate directions are still conducted independently of one another. The only difference is that every new point in each coordinate search is analysed simultaneously with each new point in all the other coordinate searches. Thus, instead of the new structure being (x_1, x_2,......, $x_i + \Delta x_i$,......., x_n) as would result from a pure coordinate search it is instead ($x_1 + \Delta x_1$, $x_2 + \Delta x_2$,....., $x_n + \Delta x_n$). The obvious benefit of this approach is that coordinate search analyses can be conducted simultaneously, thus, hopefully reducing the required number. The primary detriment is that curve fitting procedures can no longer be used since the search is no longer taking place along a unique curve.

In the simultaneous search procedure the sign of the directional derivative is the only useful piece of information available with each new analysis. Hence, instead of using curve fitting the method of bisection is employed in each coordinate search. In these directional searches the method of bisection starts with an interval which contains the minimum. This interval is then bisected and the directional derivative of f at this point determined. The sign of the directional derivative of f indicates in which half of the original interval the minimum is contained. This half becomes the new interval and the process is repeated. The procedure is stopped when a sufficiently small interval is obtained. The number of analyses in this approach is not determined by the size of the design problem but rather by the size of the initial and final intervals. Thinking in terms of moment of inertia, if the starting interval is

500 in.^{4} in length and the stopping interval 20 in.^{4} then a total of five new analyses would be required in the search. Since each of these analyses simultaneously serves all the coordinate searches, approximately five analyses are all that is required to complete the process. This should be compared to a minimum of $n + 1$ analyses for a pure coordinate descent method which, in the example of the four-story building of Figure 24 would amount to a minimum of nine analyses.

The next portion of the algorithm to be discussed, phase 2, deals with the generation of usable designs from unusable ones. A design is unusable when one or more of the system constraints is violated. Thus, the task of phase 2 is to adjust the initial design so as to satisfy the violated constraints. Since the initial design may be the unconstrained optimal, minimal satisfaction of these violated constraints is desirable. What phase 2 amounts to then is a search for the surfaces of violated constraints (as shown in Figures 25-31). Since the constraint functions display little member interdependence, that is, each is essentially dependent on only one component of the design vector, a coordinate search procedure seems appropriate for phase 2 also. In this case, however, the searches can be conducted along fixed lines so that curve fitting is feasible.

Let g represent the vector of constraint functions with components g_i where $g_i(x) = 0$ for each constraint. Then the constraint surfaces are given by $g_i(x) = 0$ for each i. Phase 2 thus involves finding the roots of nonlinear equations. Initially in the search for these roots information is available at only one point, the initial design. Assuming only function value and gradient data are supplied at this point, Newton's method must be employed to obtain the first estimate of an x which satisfies

$g_i(x) = 0$. If x^k is the present design and x^{k+1} is the estimated root then Newton's iteration formula is given by

$$x_j^{k+1} = x_j^k - g_i(x^k)/g_i'(x^k)$$

where a coordinate search along x_j is assumed and $g_i'(x^k) = \partial g_i(x^k)/\partial x_j$.

When the constraint function is evaluated at x^{k+1} information at two points is available. At this juncture the derivative estimated iteration formula [41] given by

$$x_j^{k+1} = x_j^k - \frac{g_i(x^k)}{g_i'(x^k)} - \frac{[g_i(x^k)]^2}{2[g_i'(x^k)]^3}\,\overline{g}_i''(x^k)$$

where

$$\overline{g}_i''(x^k) = -6[g_i(x^k) - g_i(x^{k-1})]/(x_j^k - x_j^{k-1})^2 + 2[2g_i'(x^k) + g_i'(x^{k-1})]/(x_j^k - x_j^{k-1})$$

can be employed. This formula can be re-used on the last two points of the sequence $\{x^k\}$ until a satisfactory estimate of the root is obtained. This apparatus is essentially what is used as phase 2 in the algorithm in the appendix with a few additional devices for special situations.

In the case where the initial design is given by the unconstrained optimal further refinement is called for. Intuitively, it would seem that the constrained optimal should lie close to the projection of the unconstrained optimal onto the violated constraint surface. Hence, for this case several additional steps have been added to the end of phase 2 which adjust the phase 2-determined usable design to the usable design given by the above projection.

Phase 3 is the design improvement portion of the automated design algorithm. A relatively simple and seemingly appropriate approach to this phase is provided by the gradient projection method [40, 42]. In this procedure the gradient of the cost function, ∇f, at the present design, x^k, is projected onto the surface of active constraints (a constraint is active when $g_i(x^k) = 0$). This projected gradient then serves as a direction vector for one iteration in the search for the optimal.

Let A be a matrix whose columns are composed of the gradients of the active constraints at x^k, that is

$$A = [\ \nabla g_i(x^k) \cdots\cdots \nabla g_j(x^k)\ ]$$

where g_i through g_j are active constraints and ∇g_i is a column vector. Then the gradient projection algorithm proceeds roughly as follows:

Assume x^k is usable, then

Step 1. Find the set of active constraints at x^k and form A.

Step 2. Compute $P = I - A\ (\ A^T\ A\ )^{-1}\ A^T$ and $d = -\ P\ \nabla f(x^k)$.

Step 3. If $d = 0$ go to step 5, otherwise find $\bar{\alpha}$ such that

$$f(x^k + \bar{\alpha}d) = \min\ \{\ f(x + \alpha d)\ \mid\ \alpha \geq 0 \text{ and } x + \alpha d \text{ is usable}\ \}.$$

Step 4. Set $x^{k+1} = x^k + \bar{\alpha}d$; $k = k + 1$ and go to step 1.

Step 5. Compute $\beta = -\ (\ A^T\ A\ )^{-1}\ A^T\ \nabla f(x^k)$.

Step 6. If $\beta_j \geq 0$ for all β_j in β then stop, x^k is the optimal, otherwise go to step 7.

Step 7. Delete the column from A corresponding to the constraint with the most negative component of β and go to step 2.

Refinements are obviously necessary in order to make this algorithm implementable. For example the "ε procedure" of [42] is employed for computational reasons [38] and to insure convergence [42]. In addition a means of establishing $\bar{\alpha}$ in step 3 is required. To this end a cubic is fit [40] through two points on the line $x + \alpha d$ with the minimum of this cubic being used to define $\bar{\alpha}$. In order to procure an $x + \bar{\alpha}d$

which is usable the constraint location apparatus of phase 2 is employed as necessary. The stop criterion in the above algorithm is that $d = 0$. In actual implementation this condition is impossible to satisfy and must be replaced with the more appropriate requirement that $d \leq \eta$ where η is a small number.

The foregoing three phases establish the essential body of the automated design algorithm used herein. A phase 4 is present at the close of the above design process in order to facilitate use of the constraint ordering idea discussed in section 6. Throughout the above three phases only primary constraints are accomodated. Hence, at the close of the design process a check must be run of all the other system constraints. This check is conducted in phase 4. If any secondary constraints are found to be violated then they are added to the primary constraint list and the algorithm returns to the start of phase 2.

This concludes the discussion of the automated design algorithm as developed and used herein. Clearly many of the operating details of this algorithm have been left out of the discussion. More information is available in the appendix as well as a program listing which is the ultimate source of programming detail.

Before closing it should be noted that derivative information for the lifetime cost and system constraints has been assumed throughout this section. While specific derivative information has not been developed herein it is quite straightforward to obtain. A fairly complete discussion of similar derivative computations can be found in [38].

9. EXAMPLES

To further explore the automated design modeling developed herein as well as generate some operating experience with the algorithm of the preceding section two example problems are investigated in this section. The first example simply represents a continuation of the investigation into the four-story frame of section 7. The second example is an eight-story, single bay frame.

9.1 Four Story Frame

It was speculated in section 5.2.4 that the expected damage profile for a one story frame (see Fig. 22) should resemble in shape the damage profiles for multistory frames. It would be interesting to examine this possibility in light of the four story optimal frame of Fig. 24 and equation (7.1). Hence, the damage profile curve for this structure is produced and shown in Fig. 32. The discontinuity in this curve at 0.25 g results from employing 5% of critical damping for earthquakes of less than 0.25 g and 10% of critical damping for earthquakes larger than 0.25 g. Recall that 5% of critical damping was assumed for all earthquakes for the one-story frame.

Aside from the discontinuity, the curve of Fig. 32 is strikingly similar to that of Fig. 22. To determine how similar, equation (5.10) can be scaled to yield the same peak value as the four-story curve of Fig. 32. Plotting the resultant equation over the four-story curve it is found that the two coincide identically up to the discontinuity. Alternatively, scaling equation (5.10) to yield the four story damage value at 0.3 g shows this equation to match the curve of Fig. 32 identically beyond 0.25 g. It is clear then that the single-story and the multistory frames have damage profiles identical in form and differing only in magnitude. The ramifications of this result are significant. They imply that integration of multistory damage profiles can be accomplished using a single integration point and a "universal" single story damage curve. Since it does not matter which earthquake is chosen as the integration point the design earthquakes of section 6.2 could be used so that no additional analyses would be needed. Thus, what could have amounted to a very expensive numerical integration may be reducible to an almost "something for nothing" situation.

Acceptance of this conclusion obviously awaits additional verification, either numerical, such as used here, or analytical. In addition, it would be interesting to see if the principle can be generalized to include a broader category of frames than just those with a single bay.

Because of the discontinuity in the curve of Fig. 32, the exact integration points determined from equation (5.10) are no longer applicable. Exact integration points for the trapezoidal approach to the four story damage curve are (0, 0.065, 0.309, 0.5). Using these values the four-story optimal is recomputed to be

$$X = (670, 660, 2377, 641, 1894, 710, 2003, 1039)^T \qquad (9.1)$$

where compact sections and fully rigid connections are assumed. The change from the previous optimal (equation (7.1)) is not large but it is significant, indicating the importance of reasonably accurate integration. As far as the nature of the optimal solution is concerned the second story column strong earthquake ductility demand limit remains the only active constraint in the new optimal, with all other constraints far from active. Most significantly the beam deflection and column buckling constraints are essentially of no concern.

Neither are the strong earthquake beam ductility restrictions. Otherwise, as groups, the remainder of the constraints seem to be reasonably competitive depending upon the chosen allowables. The ductility demands on the optimal structure for the strong earthquake are (top story down)

Beams	Columns
0.69	0.58
0.66	0.88
0.99	1.00
1.12	0.98

Note that the first story column ductility demand is only 0.98 even though the allowable is 2. This is a good indication of the strong influence of the cost function on the columns. The computed lifetime cost for the optimal frame is \$4465.75 with a construction cost of \$2158.33.

A comparison between the above minimum LC frame and the minimum construction cost frame could prove to be of interest. Hence, the minimum construction cost frame is computed and found to be

$$X = (318,\ 245,\ 818,\ 480,\ 1079,\ 587,\ 1204,\ 516)^T \ . \qquad (9.2)$$

This structure is substantially more flexible than that of equation (9.1). This increased flexibility is clearly reflected in the strong earthquake ductility demands which are now

Beams	Columns
1.10	1.10
1.20	1.00
1.27	1.00
1.33	1.59

and show a significant increase over the minimum LC frame demands.

As is typical for minimum construction cost (weight) frames the design of equation (9.2) is fully constrained (i.e., one constraint is active for each component of the design vector). The roof beam is confined by the static moment constraint (6.2) with the remaining beams limited by the moderate earthquake moment constraint (6.18). The first story column is constrained by the moderate earthquake moment inequality (6.22) for the bottom end moment. The second and third story columns are confined by the strong earthquake ductility limit (6.22) for the top of the column. The fourth story column is limited by the static moment inequality (6.6) for the top of the column. The beam deflection, column buckling and strong earthquake beam ductility limits still remain of little concern even for this relatively flexible frame. The construction cost for this frame is $1554.71 which represents a 28% reduction from the limimum LC frame construction cost. The LC is now $4905.46, however, which represents a 10% increase in LC over that of equation (9.1).

The designs of (9.1) and (9.2) both represent acceptable design practice, they are strikingly different, however, and reflect a very real difference in design philosophy. This choice of design philosophy is clearly one which should be addressed by every prospective building sponsor prior to design formulation.

9.2 Eight-Story Frame

To see what effect frame height has, if any, on the results ascertained to this point an eight-story frame is now examined. This frame is identical to the four-story structure introduced in section 7 as far as bay width, story heights, loading, costs, etc. are concerned.

The only difference is that eight stories are involved instead of four. The floor values are also changed somewhat with a top floor value of \$237.78 and all other floors valued at \$211.36.

The optimal design determined via the algorithm of section 8 is

$$X = (448,\ 513,\ 1739,\ 583,\ 1805,\ 631,\ 1986,\ 672,\ 2105,\ 739,\ 2021,\ 847,\ 1733,\ 916,\ 1775,\ 1007)^T$$

Four constraints are active at the optimal. They are the strong earthquake ductility limit (6.22) for the top ends of the second, third and fourth story columns and the bottom end of the second story column. Once again the beam deflection, column buckling and beam ductility constraints are far from active. The strong earthquake ductility demands are (top story down)

Beams	Columns
0.81	0.65
0.68	0.75
0.81	0.84
0.89	0.93
0.97	1.00
1.10	1.00
1.29	1.00
1.31	1.27

Note that while a ductility allowable of 2 is specified for the first story column the ductility demand is only 1.27. This situation is similar to the four-story result and would suggest that while several strong earthquake column ductility allowables are reached these constraints do not represent strong limits on the design process. Indeed, if the column ductility allowables are relaxed to 2 then it seems quite likely that a completely unconstrained optimal would result. This conclusion is supported by the illustration in Fig. 26 wherein it can be clearly seen that the unconstrained optimal lies just below the surface of column ductility limit 81. If column ductilities of 2 are allowed, and there appears to be clear evidence that they could be [24, 31], then only the relatively simple procedures of phase 1 of the automated design algorithm would be required to develop a structural design from scratch. This is a very interesting and potentially money-saving prospect which deserves further investigation.

This concludes the structural design portion of the examples section. While brief, these examples have lent some support to a few of the major conclusions which have been drawn thus far. What is obviously required, however, is a much broader study of more general frame configurations, loading conditions and system constraints. The need for such an investigation hinges of course on the acceptability of the basic premise of this effort, namely the usefulness of the lifetime cost approach to design.

9.3 Algorithm Performance

A few words about the design algorithms performance probably are in order. Not because the algorithm performed outstandingly but simply because it worked and thus can serve as a baseline for future algorithm developments. This discussion must be taken in light of the fact that the computer code which was built around the design approach outlined in section 8 does not represent a numerical masterpiece. Rather it is composed essentially of off-the-shelf items, usually in their most rudimentary forms. Virtually no effort was put into trying to speed up convergence or improve efficiency. With these comments in mind the performance of the automated design algorithm as it developed the eight story optimal is presented in the following.

The starting vector for the algorithm was X = 1000 u where u is the unit vector (vector of ones). The performance of the algorithm in terms of LC versus the number of analyses completed is given in Fig. 33. As shown, the plot is divided into the three phases outlined in section 8. There are several interesting items in this illustration worth noting.

First, only nine analyses were required to complete phase 1 despite a poor initial design. Included in these nine analyses is not only the interval bisection apparatus but also an interval establishment procedure as well. Thus, it would appear that the simultaneous coordinate search procedure does indeed work quite well.

The next item to note is the relatively small difference between the phase 1 and phase 3 results. This is a clear indication of the relatively close proximity of the constrained and unconstrained optimals. Equally as important to observe is the very small difference between the phase 2 and phase 3 results. The intuitive feeling that projection of the unconstrained optimal onto the violated constraint surface would result in a good estimate of the constrained optimal appears to be well-founded. Very little modification of phase 2 results was necessary in phase 3. This raises the question of why so many analyses were required to complete phase 3. The answer to this is two-fold. First, it is a reflection on the poor convergence rate of the method of steepest descent, which is the technique used in the active constraint space (projected gradient space) to pursue the optimal (see step 3 in the gradient projection algorithm of section 8). As can be seen in Figs. 30 and 31, the design space is somewhat ill-conditioned. This situation carries over into the active constraint space. The method of steepest descent is a poor choice under these conditions. Better use of the projected gradient in the active constraint space would most certainly shorten phase 3.

Another probable cause for the longevity of phase 3 is what appears to be an unrealistically small stop criterion (the η in $d \leq \eta$). The criterion used herein was chosen with investigation in mind, rather than design, and thus is probably smaller than what would be employed in a design office.

One item of note which is only implicitly reflected in Fig. 33 is the superb performance of the constraint function search apparatus presented in the phase 2 discussion in section 8. Despite repeated tests of these procedures at sometimes considerable distances from violated constraint surfaces, more than two iterations were never required and in actual operation one iteration usually proved to be sufficient. This is in direct contrast to the difficulties encountered in [38] using alternative procedures and it is hard to imagine how this performance could be improved upon.

Before closing it should be noted that the overall number of required analyses depended greatly on the numerical tolerances which were specified for the algorithm. These tolerances are given as recommended values in the appendix and should be accommodated in any future algorithmic comparisons.

10. SENSITIVITY

The sensitivity of a problem solution to the various system parameters involved is always an interesting and useful exercise to perform. Not only does it provide an indication of how the system parameters interplay in determining a solution but it also provides an estimate of where and how much error can be tolerated in these values. Both aspects of solution sensitivity will be addressed in the following.

With the exception of the single active constraint, all of the quoted sensitivities are developed numerically on the basis of the optimal four story structure presented in the previous section. Thus, the numerical tolerances used in the automated design algorithm must be accounted for in any given sensitivity.

The discussion is divided according to the origin of the system parameter to be explored and includes parameters related to construction cost, damage estimates and structural loading.

10.1 Construction Cost

Relative to construction the most important cost is that of steel for the structural members. To investigate the sensitivity of the optimal design to the cost of steel a 10% increase in this cost is imposed and a new optimal obtained. The result is a significant shift in the location of the optimal with a mean change (absolute) in the moment of inertia of the beams of 81 in^4 and in the columns of 48 in^4 (excluding the second story column, X_6, which is constrained). The LC for the optimal design increases 3.92%. This equates to an approximate sensitivity of the LC to the cost of steel of 135 ($LC/$ per cwt of steel).

As far as design error is concerned the LC at the original optimal increases 3.97% so that the error in LC at the former optimal as compared to the new optimal is only 0.051%. Thus, while the LC is quite sensitive to changes in the price of steel the construction of a building which costs only 0.051% of the LC more than the optimal is hardly a serious matter. Hence, if an accurate determination of the LC is important then the price of steel had best be accurate. However, if determination of the optimal design is important a 10% error in the price of steel seems quite tolerable.

The next item of interest relative to construction cost is the size-extra charge. It is expected that the optimal design will be relatively insensitive to this charge. Hence, the primary interest here is in examining the model itself rather than its effects.

Since the sensitivity of the optimal to a change in size-extra charge is likely to be lost in the numerical tolerances of the automated design algorithm this sensitivity is examined in light of the cost of steel results. The mean size-extra charge assessed for the four story optimal is 0.49 ($/cwt of member steel). Hence, a 10% increase in this rate represents a 0.05 ($/cwt of member steel) increase in construction cost. Using the sensitivity computed for the cost of steel this increase results in a change in LC of $6.73 and a mean change in member sizes of 3.1 in^4 for beams and 1.8 in^4 for columns. These values are clearly insignificant.

Despite the insensitivity of the optimal design to the size-extra charge this model should not be dropped entirely, however, since in total cost (about $0.50/cwt of member steel) it is significant. What is indicated by the above results is that the power curve approach to this cost as discussed in section 5.1.1 is probably not warranted. Rather, a mean value model seems more in keeping with the sensitivity of the solution to this cost. Thus, for the four story design problem a size extra charge of 0.49 ($/cwt of member steel) would represent a good model. Clearly with mean value modeling the size-extra charge could be added directly into the cost of steel for the members.

Consider now the connection cost model of section 5.1.2. Imposing a 10% increase in this cost and recomputing the four story optimal results in virtually no change. This is another case where the solution sensitivity is lost in numerical tolerance. To determine the significance of this model it is dropped and the optimal again computed. The result is a mean change in beam size of 282 in^4 and in column size of 12 in^4 (excluding X_6). These values represent significant readjustment of the optimal design and rule out disposing of the connection cost models. The insensitivity of the optimal design to large changes (10%) in connection cost indicate that sophisticated modeling is not necessary. What has been developed herein is probably sufficient and could possibly even be simplified.

The last item to consider with respect to construction is the cost of painting. Again numerical investigation reveals a solution sensitivity less than the numerical tolerances being employed. Dropping the cost of painting model results in a mean change in the optimal design of 79 in^4 in the beams and 4 in^4 in the columns (excluding X_6). This alteration is significant but only marginally so and suggests that only rudimentary modeling of the cost of painting is required. The modeling developed herein appears to be much more than is necessary.

10.2 Damage Estimate

There are several damage estimation parameters of interest with regard to sensitivity. They include the slope of the damage curve, the nonstructural value of the floors, the design life and the building site seismicity. Fortunately because of the form of the damage modeling the effect of changes in each of these parameters can be investigated simultaneously. A 10% increase in any of these parameters produces exactly the same result in terms of the optimal design. Hence, the most convenient parameter is selected for evaluation.

Instigating a 10% increase in the design life of the four story structure of section 7 results in a 5.03% increase in the LC for the optimal design. The shift in the location of the optimal design vector is reflected by a mean change in the beams of 170 in^4 and in the columns of 53 in^4 (excluding X_6). The sensitivity of the LC to design life is 45 ($LC/year design life).

At the original optimal the LC increases by 5.17%. This is 0.14% higher than the LC at the new optimal. In terms of LC then the design error which results from a 10% error in design life is not significant. On the other hand if the actual value of LC is important then a 10% error in design life is quite significant.

As far as the other damage estimation parameters mentioned above are concerned, with the exception of the quoted sensitivity value, all of the preceding results are directly applicable. The general sensitivity for these remaining parameters is 22.4 ($LC/percent increase).

One difficulty with the damage estimation parameters which was not significant with respect to construction costs is the possibility of combined errors. Since the damage parameters combine in the damage model through multiplication a 10% error in each of two parameters could possibly result in a 21% overall error in the damage estimates. Thus, when assigning values to the various parameters care should be exercised to insure that potential errors cancel as much as possible.

10.3 Loading

In this category the principal parameter of interest is the distributed loading on the beams. This beam loading supplies not only the static loads but, via the mass matrix, the dynamic loads as well.

To investigate the effects of this system parameter a 10% increase in the distributed loads on the beams is initiated. As expected the optimal design shifts its location in the design space. The mean change in the beam moments of inertia is 91 in^4 and in the columns 26 in^4 (excluding X_6). The LC of the optimal increases 2.64%. The sensitivity of the LC to the beam loading is thus 11.8 ($LC/percent increase in beam loading).

Note that in the face of a 10% increase in beam loading the original optimal no longer represents a usable design. In fact X_6 must be increased by 64 in^4 in order to satisfy the associated strong earthquake ductility limit.

The final item of concern with respect to solution sensitivity is the single active constraint at the optimal design for the four-story frame. According to the Sensitivity Theorem (see section 10.6 in [40]) the sensitivity of the LC to any active constraint at the optimal design is equal to the negative of the associated Lagrange multiplier. The Lagrange multipliers at the optimal are the components of β in the gradient projection algorithm of section 8. Hence, the LC sensitivity to the strong earthquake ductility allowable of X_6 is -0.00994

($LC/kip-in).

For comparative purposes a complete tabulation of the results of this section can be found in Fig. 34.

11. CONCLUSION

There have been numerous deductions drawn throughout the preceding text. In an effort to present a concise overview of the work the major results are collected here along with some additional conclusions.

- The primary emphasis in this work has been on defining and exploring the problem rather than on the complete solution. This emphasis is clearly reflected in the amount of attention given to developing an option search procedure versus that given to option set and decision motivator modeling. From the results it is very clear that this approach is not only warranted but preferable to the reverse method (i.e., develop a solution method, then find a problem). As is the case here, very frequently a problem will yield to one mode of solution much more readily than to another. If advantage is to be taken of this, thorough formulation and exploration of the problem prior to selecting a plan of attack is essential.
- The basic format used in exploring the automated design problem has been through use of a very rudimentary design theory (section 1.1). While this theory is essentially clerical, it has proven to be very helpful in evaluating the total design problem rather than just focusing on the part which can be readily solved. If additional structure can be added to this theory, it could prove to be even more helpful to execution of a rational design process.

• The underlying design philosophy employed has been to minimize lifetime costs rather than just initial costs. This effort has shown in a limited fashion the potential viability of this design philosophy. It has also been shown that the lifetime cost approach does indeed result in a distinctly different design alternative to standard minimum construction cost (weight) procedures. Hence, a definite choice is available with the LC approach, deserving serious consideration in this context.

• The connection models used herein are somewhat limited. Based on these models, however, it would appear that fully rigid connections represent the most economical approach to frame design in seismically active regions.

• Suboptimization over member option sets can be employed successfully to reduce the number of required design variables while still retaining the global optimum within the reduced sets. Caution must be observed when utilizing this procedure, however, since it is possible to exclude the global optimum from the reduced option sets in some cases.

• For the class of design problems considered, member sizing decisions are nearly uncoupled. This fact is reflected by both the decision motivator and system constraints. Thus, very simple coordinate search procedures can be used to find approximate locations for both the unconstrained and constrained optimals. In this regard the simultaneous coordinate search technique employed here to find the unconstrained optimal is found to work very well and appears to hold promise for possible improvement. Note also that the approximate constrained optimal (the result of phase 2) requires very little alteration in phase 3. Thus, in light of the sensitivity results of the previous section it appears very likely that for design office purposes this approximate optimal could be utilized as a final design.

- At the start of this program it appeared that the most prohibitive part of the lifetime cost approach would be the expense of computing the damage cost estimate. The discovery, however, of the very promising possibility that expected damage profiles are invariant in form with respect to a structures size has potentially reduced this computation to an almost insignificant level. Further exploration of this prospect is most assuredly warranted.

- In section 6 a preliminary categorization of constraints is established with constraints labeled as either primary or secondary according to their importance in the design process. In all of the numerical computations completed in this work the constraint order established in section 6 was never violated and thus appears to be very reliable. In addition, further refinement of the constraint order is possible. Indeed it seems that the only constraints that warrant primary status are the strong column-weak girder requirements in which column ductility demands are limited to one. In lieu of this, at the very least, beam deflection, column buckling, and beam ductility limits should be placed in secondary status.

- Regarding system sensitivity it is clear that the most crucial system parameters are those associated with damage estimation. Not only is the lifetime cost at the optimal most sensitive to these values but there are several parameters in this category which act collectively. Further compounding the problem is the fact that among all the system parameters those associated with damage estimation are the most uncertain. Hence, caution should be used in establishing these values for structural design problems.

Before closing it is appropriate that some comment on future research be made. From the preceding comments it is obvious that the area requiring the most attention is lifetime damage estimation. As far as deterministic assessment is concerned there is essentially nothing available. In light of the apparent viability of the lifetime cost approach it would appear that this area is wide open for research.

With reference to the design problem only a very small portion of the overall design process has been addressed here or elsewhere. There are many more decisions within this process which could be automated and many more options which should be considered. A substantial amount of work thus remains in the problem description area. The conclusions reached herein indicate that this area should be thoroughly investigated before any substantial effort is mounted to develop additional design automation algorithms.

As far as application is concerned one item is of note. The refurbishing of existing buildings to meet present earthquake standards is a frequent, major and expensive undertaking. In the way of analytical tools, little appears to be available to assist in the retrofitting of older structures, however. Some of the methodology developed herein would seem to be applicable to this task, particularly optimal retrofitting (refurbishment at minimum cost), and thus could provide a beginning for a developmental effort in this area.

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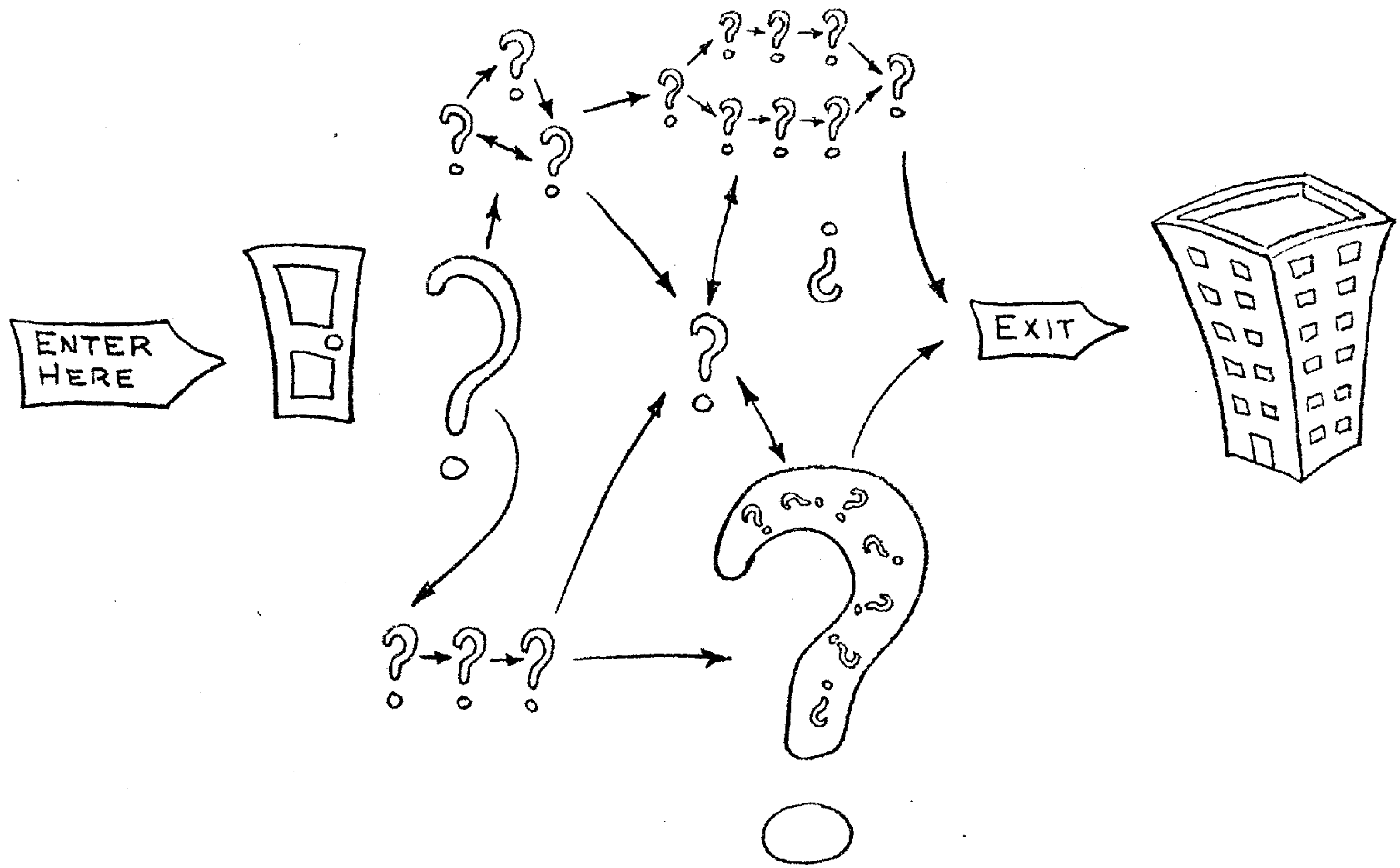


FIGURE 1. THE DESIGN PROCESS

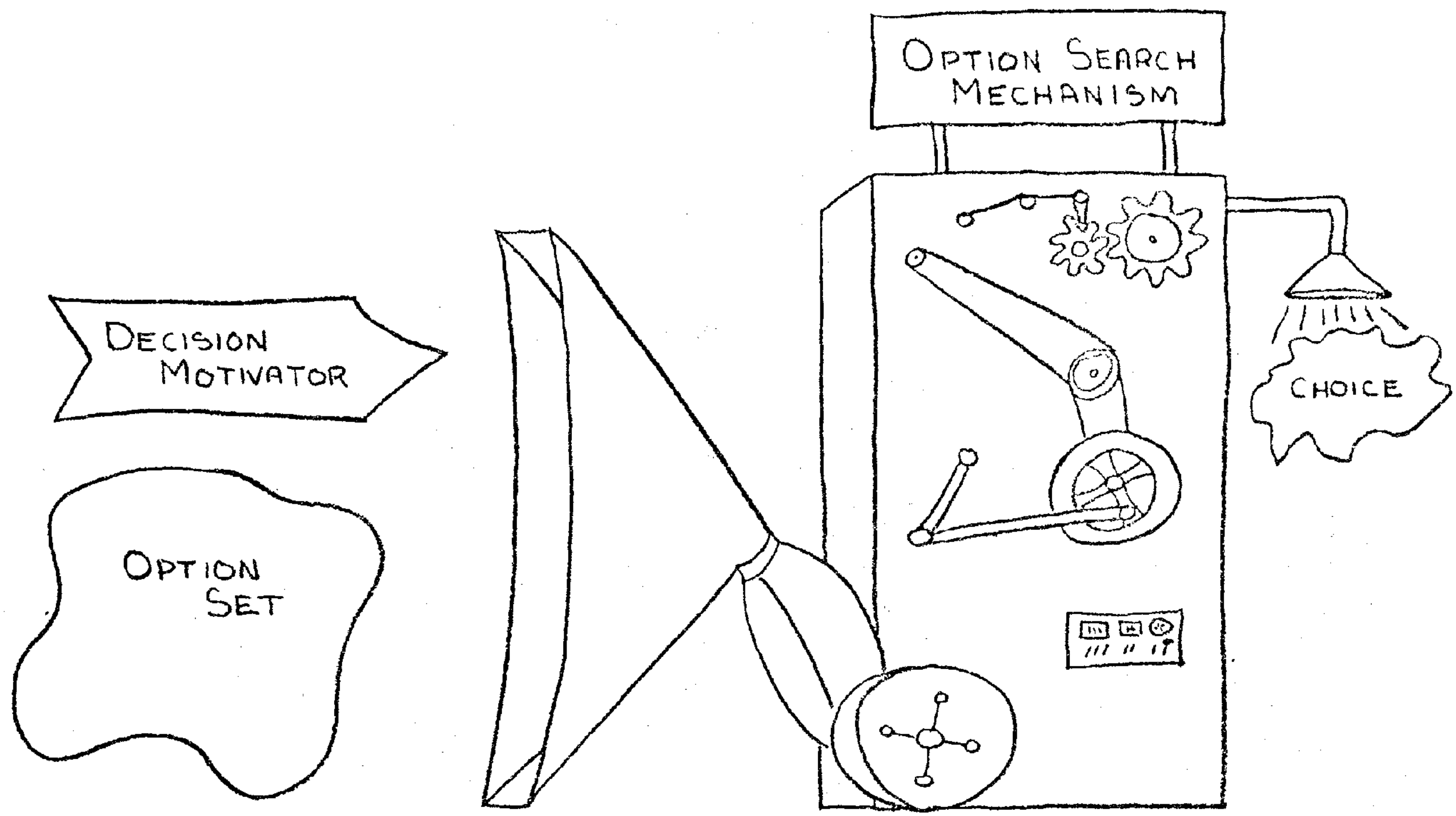


FIGURE 2. THE DECISION PROCESS

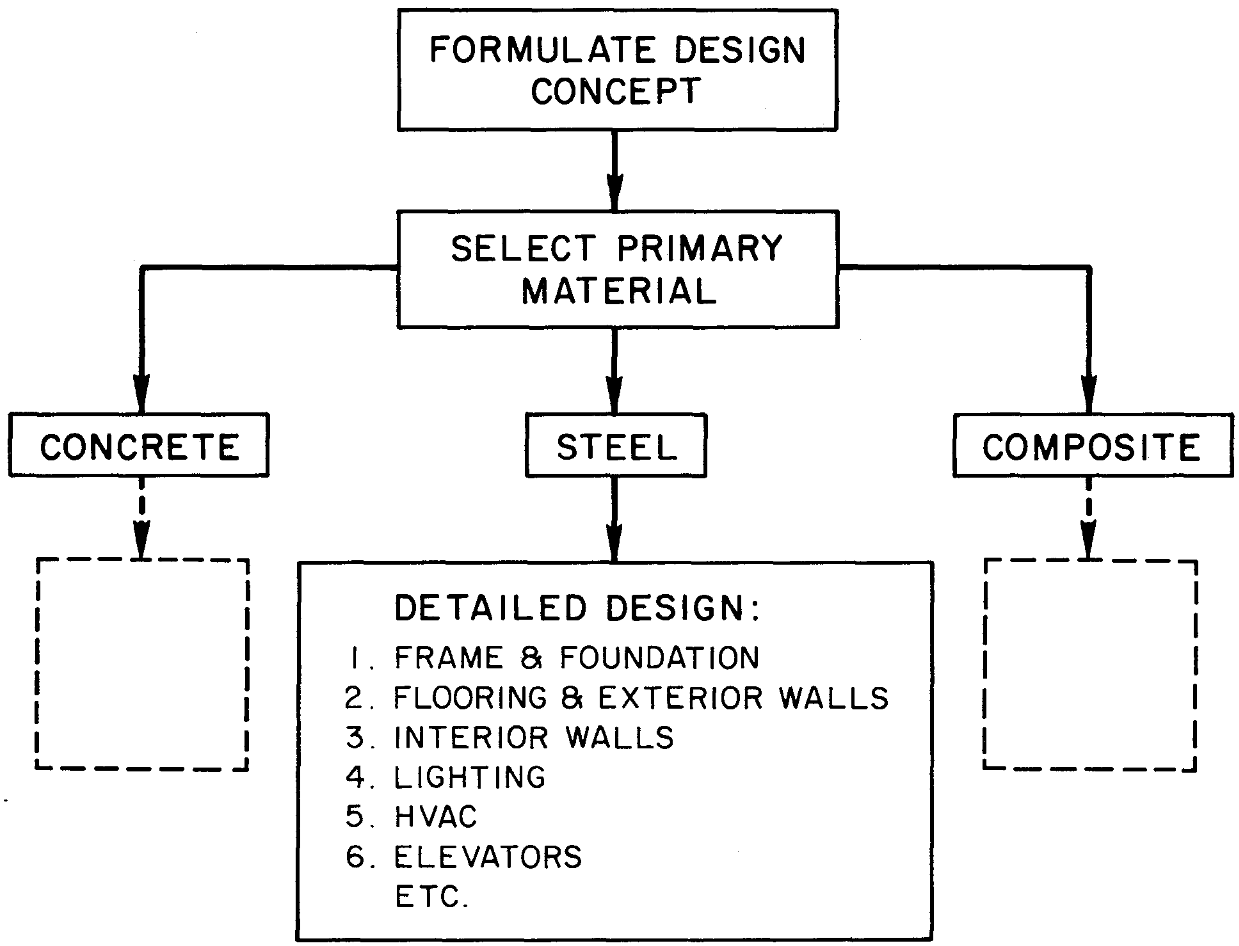


FIGURE 3. DESIGN OF BUILDINGS

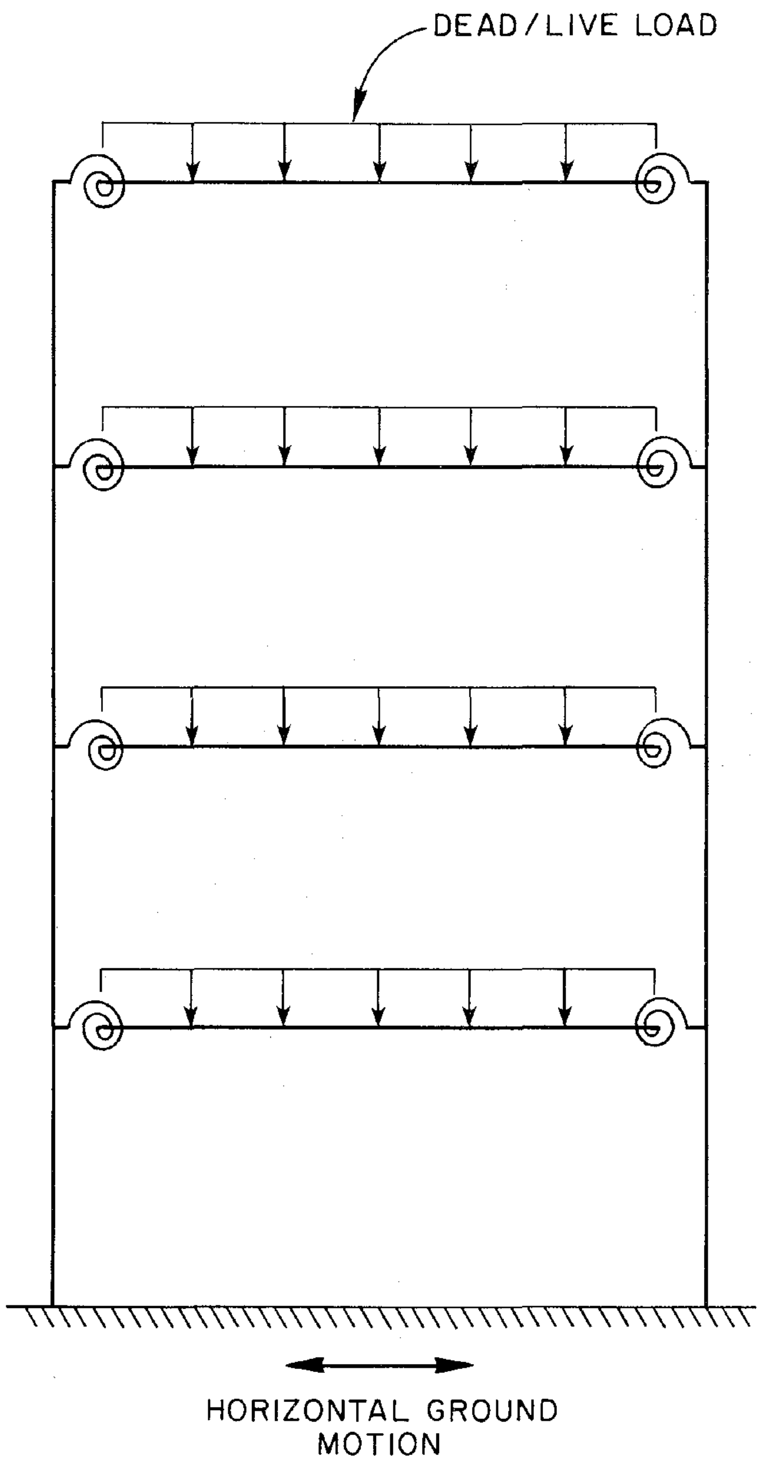


FIGURE 4. STRUCTURE WITH LOADS

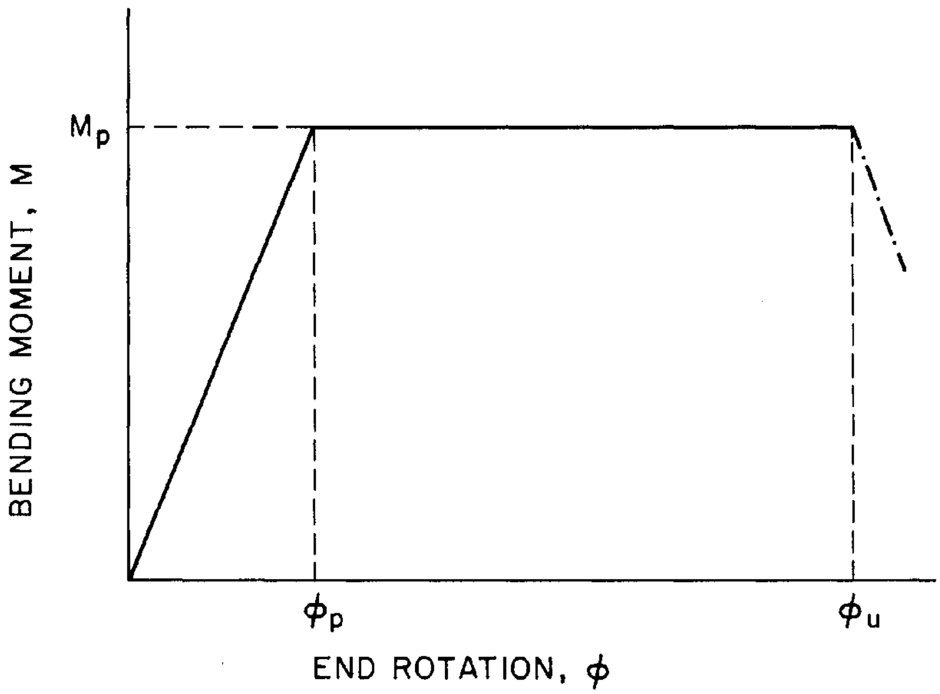


FIGURE 5. MEMBER MOMENT-ROTATION CURVE

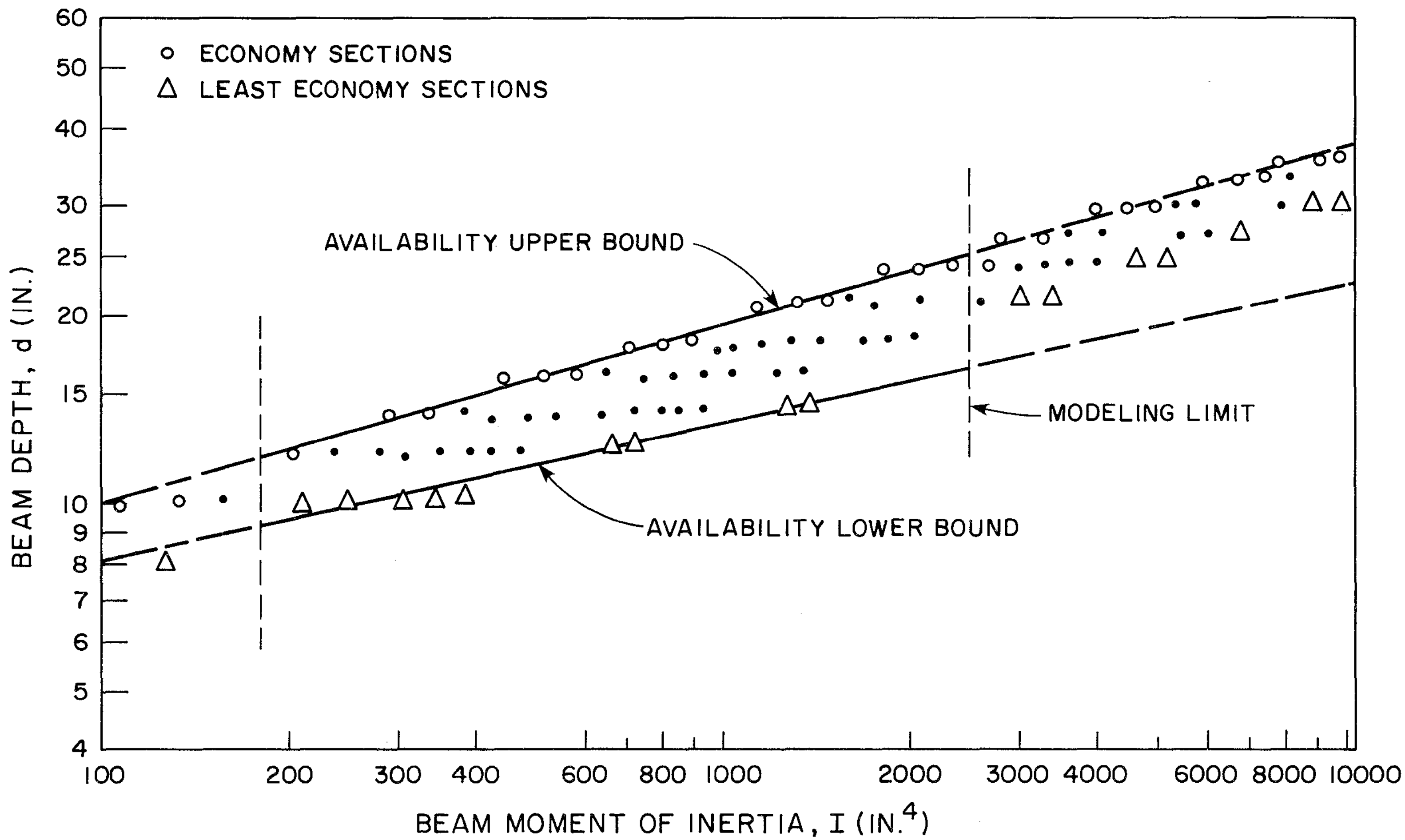


FIGURE 6. BEAM OPTION SET

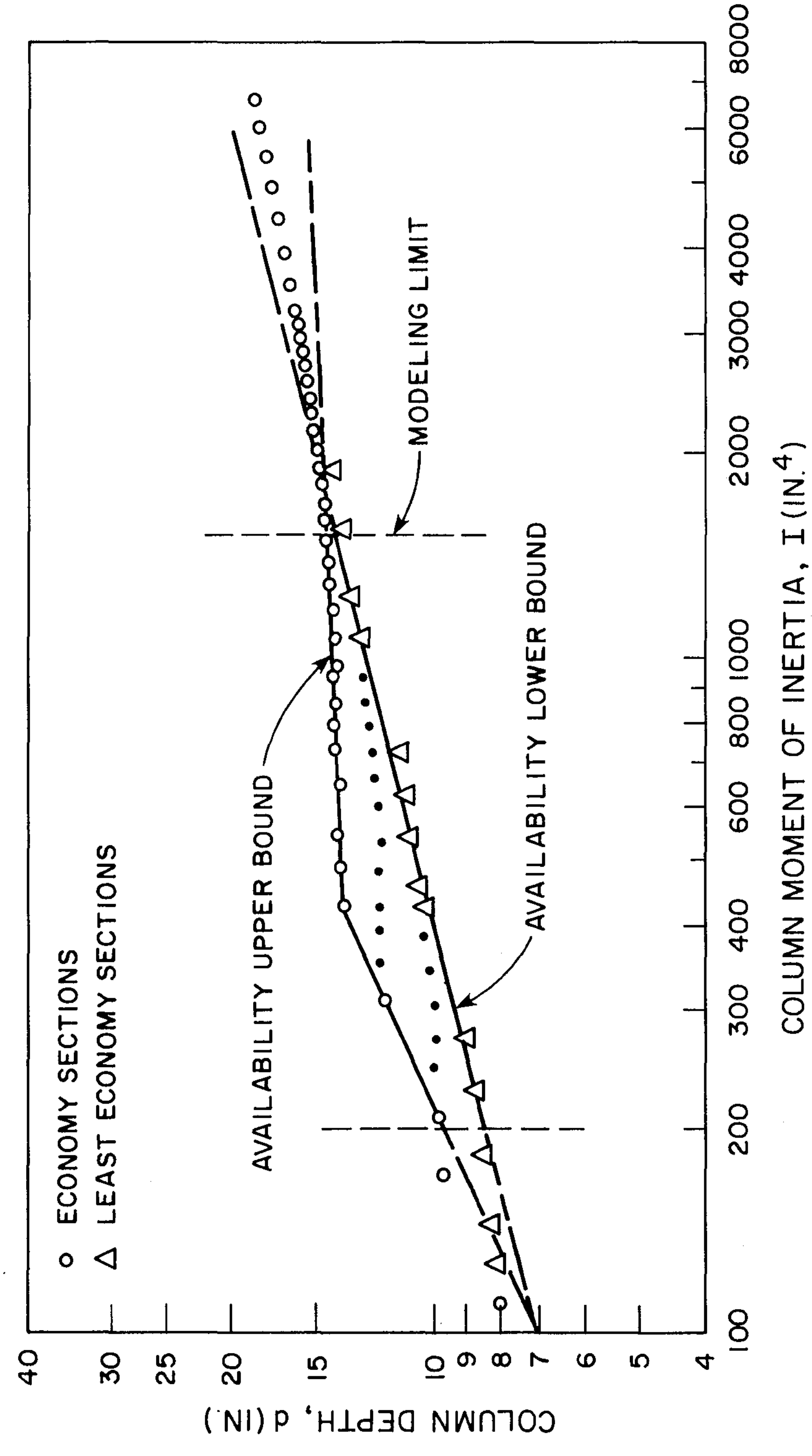


FIGURE 7. COLUMN OPTION SET

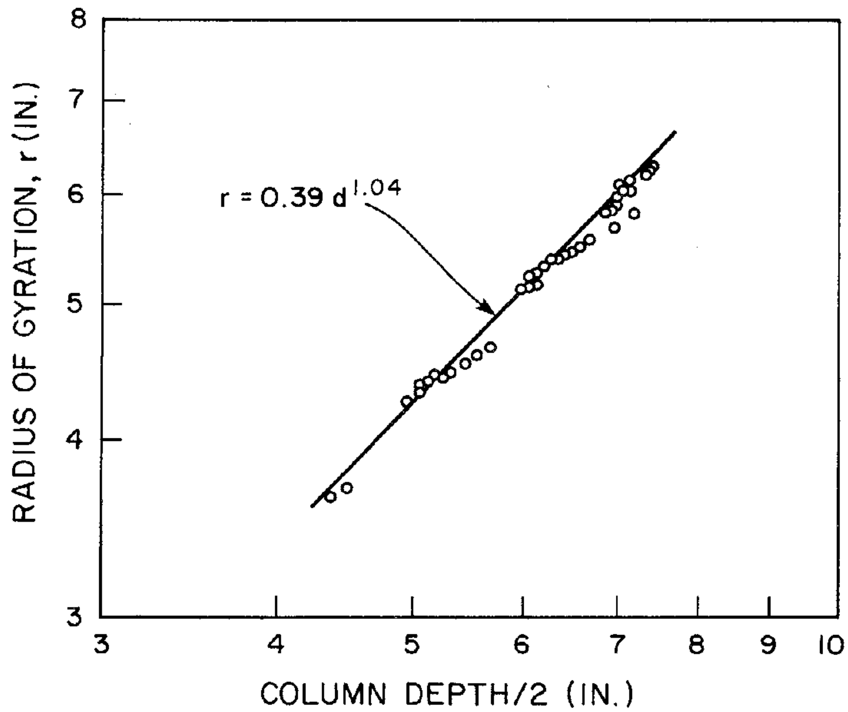


FIGURE 8. RADIUS OF GYRATION MODEL FOR COLUMNS

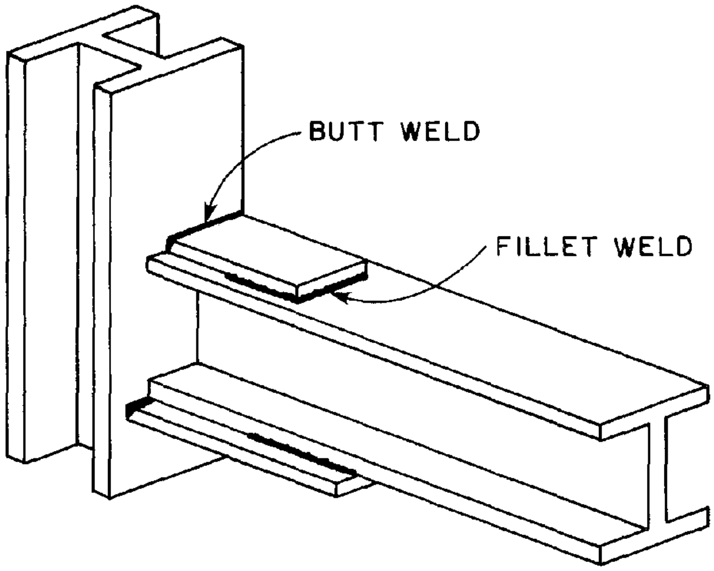


FIGURE 9. WELDED TOP AND BOTTOM PLATE CONNECTION

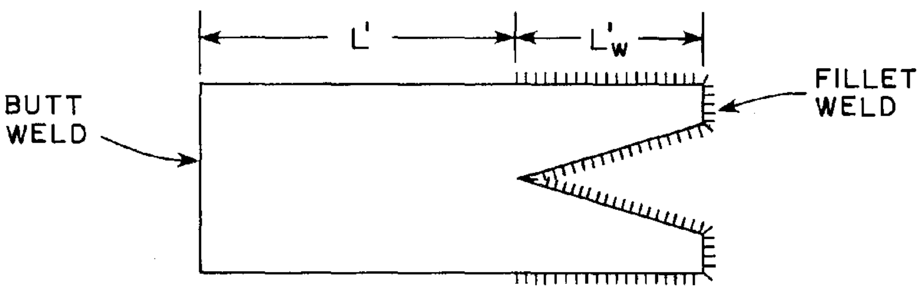


FIGURE 10. TOP PLATE DETAIL

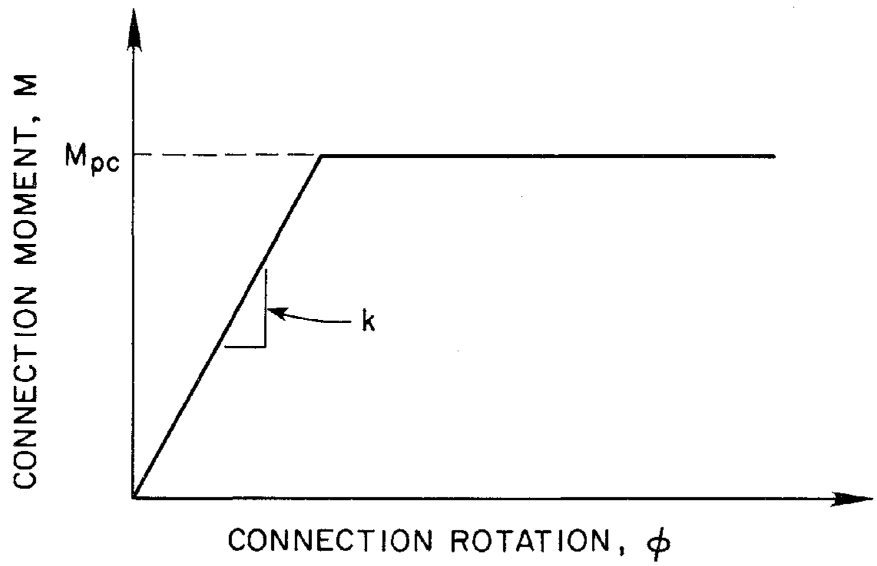


FIGURE 11. CONNECTION MOMENT-ROTATION CURVE

AREA (IN.)	32	30	28	26	22	18	14	10	8
R (%)	48.6	49.4	50.3	51.2	53.3	55.8	58.8	62.8	65.4

FIGURE 12. CONNECTION RIGIDITY VERSUS BEAM AREA

SIZE (IN)	WEIGHT (LB/FT)	EXTRA CHARGE ($/cwt)
W36	230 - 300	0.55
W36	135 - 194	0.50
W30	172 - 210	0.50
W30	99 - 132	0.45
W24	68 - 160	0.45
W24	55 - 61	0.60
W21	55 - 142	0.45
W18	64 - 114	0.45
W18	35 - 40	0.65
W16	58 - 96	0.45
W16	36 - 50	0.55
W14	142 - 426	0.45
W14	61 - 136	0.45
W12	65 - 190	0.45
W12	40 - 58	0.50
W12	27 - 36	0.60
W10	49 - 112	0.50
W10	21 - 29	0.75
W10	15 - 19	1.10
W8	31 - 67	0.55
W8	17 - 20	0.90
W6	12 - 16	1.60

FIGURE 13. SIZE EXTRA CHARGES FOR WIDE FLANGE SECTIONS [7]

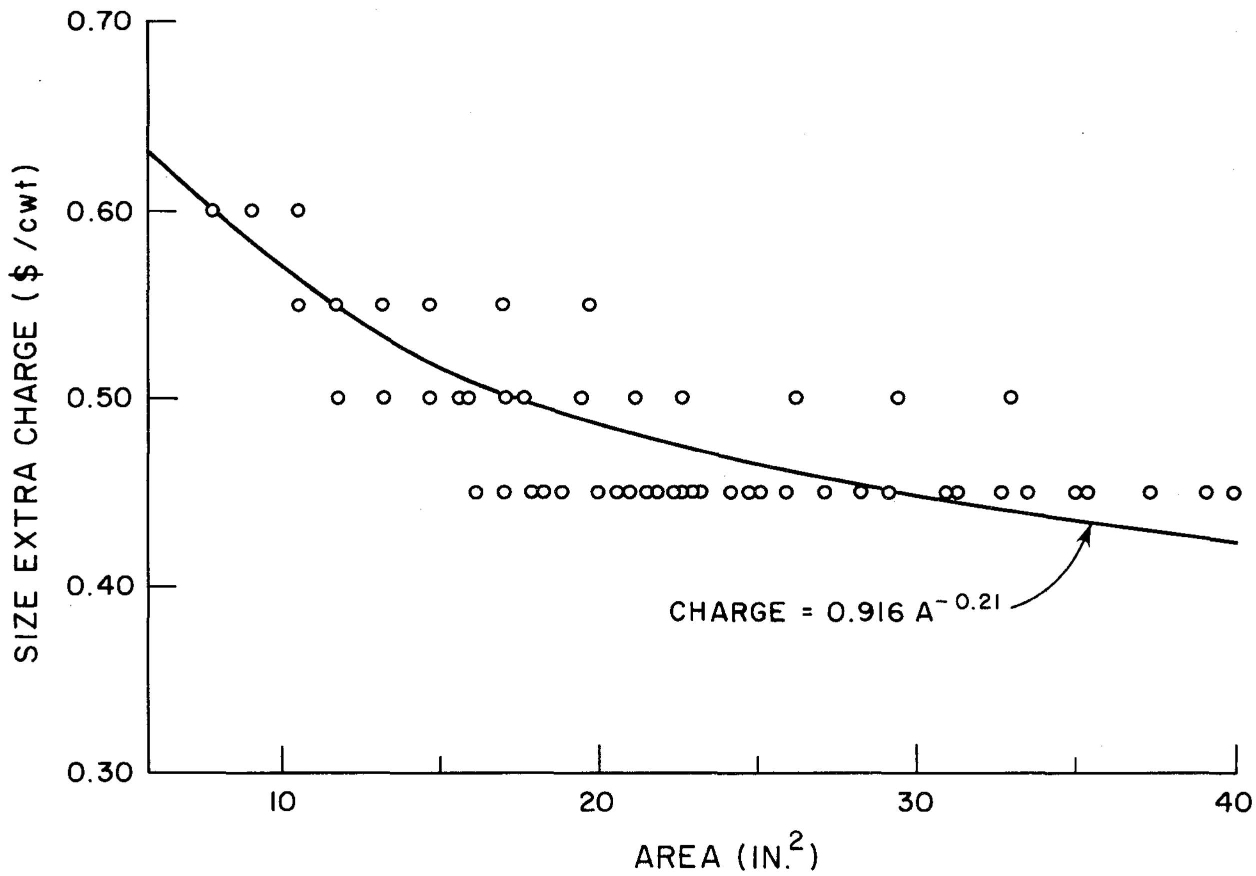


FIGURE 14. SIZE EXTRA CHARGE VERSUS AREA FOR SELECTED WIDE FLANGE SECTIONS

QUANTITY	EXTRA CHARGE (PER CWT)
4000 LB AND OVER	NONE
2000 LB TO 3999 LB	$0.25
1000 LB TO 1999 LB	0.75
UNDER 1000 LB	2.25

FIGURE 15. QUANTITY EXTRA CHARGE FOR STEEL [7]

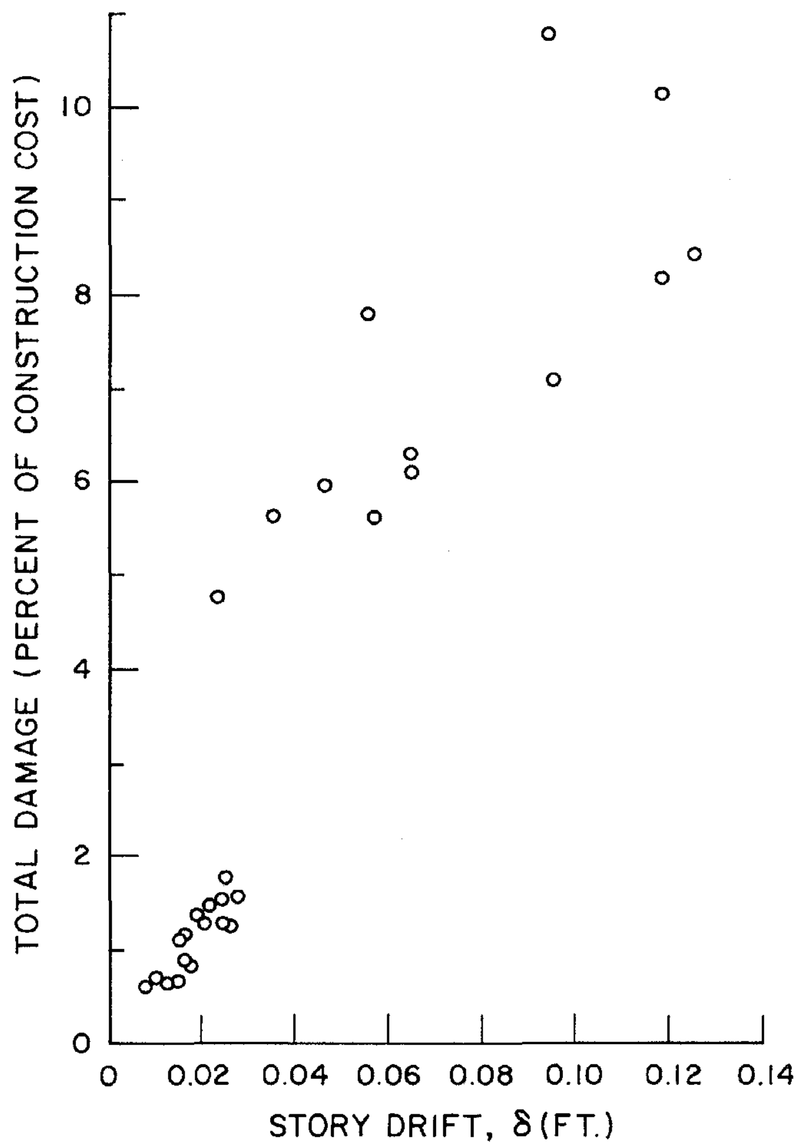


FIGURE 16. TOTAL DAMAGE VERSUS STORY DRIFT [29]

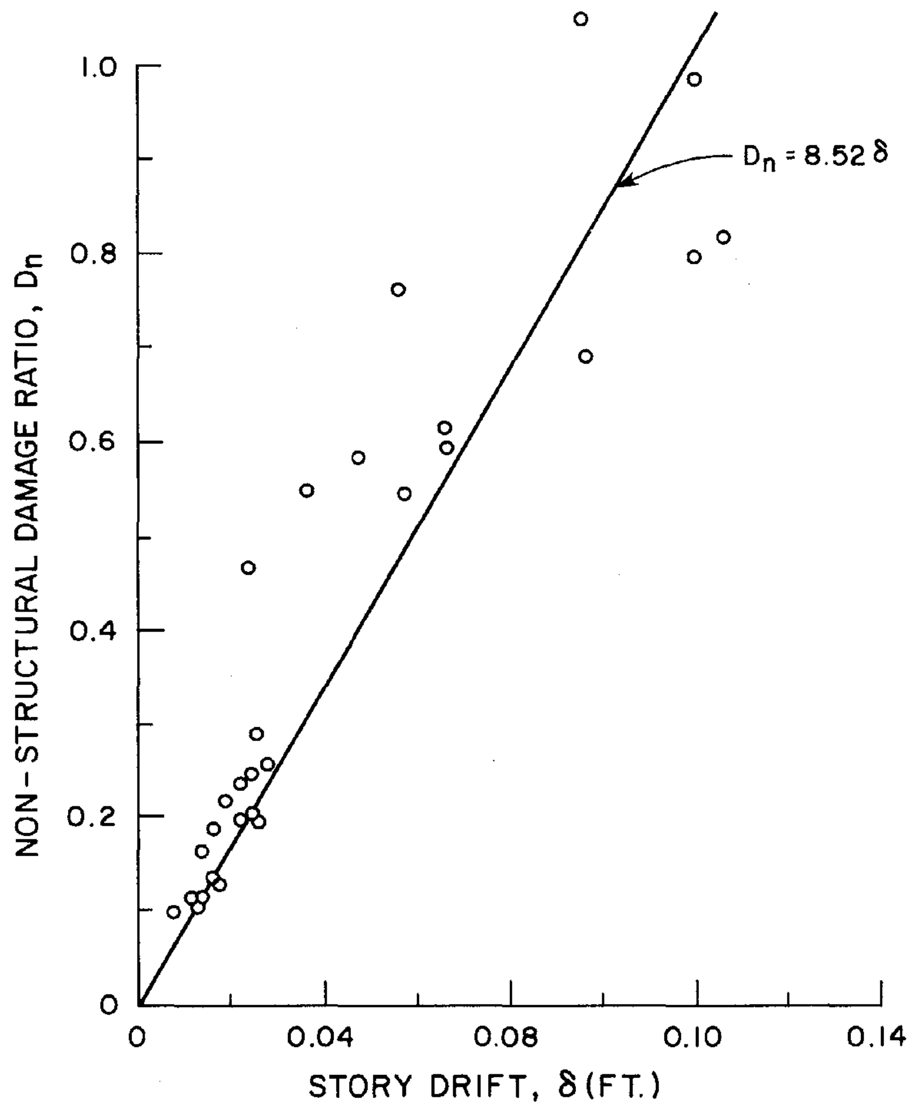


FIGURE 17. DAMAGE RATIO VERSUS STORY DRIFT

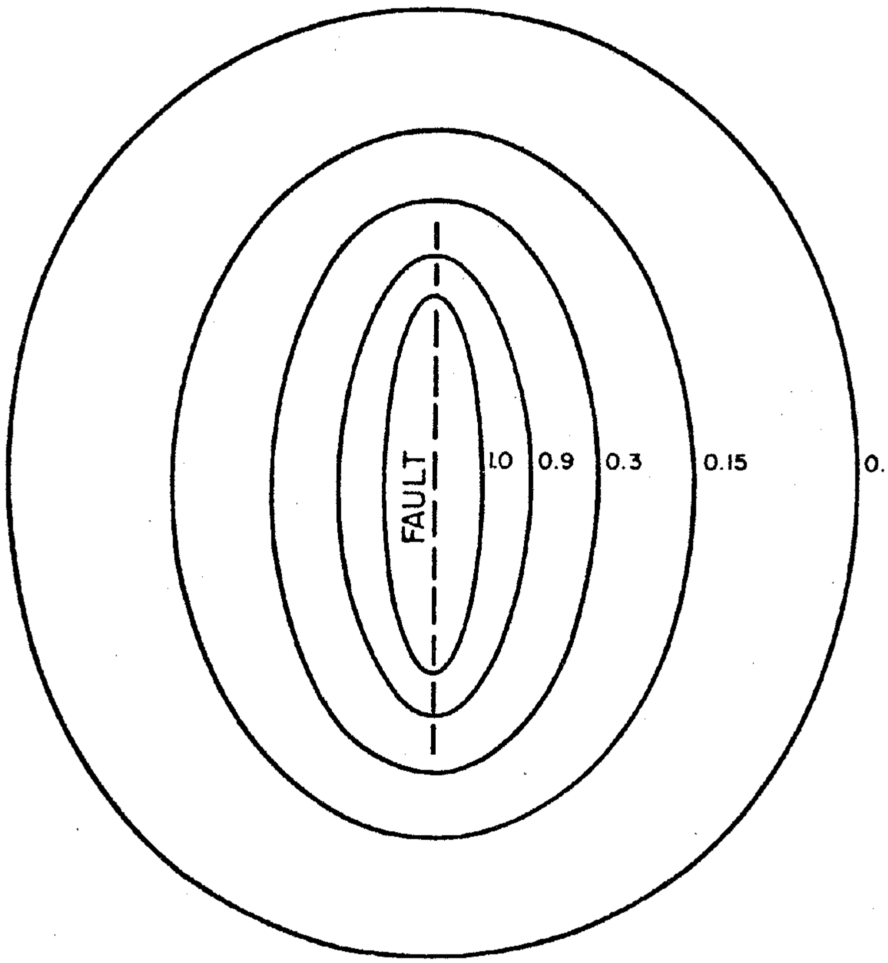


FIGURE 18. INTENSITY ATTENUATION PROFILES [37]

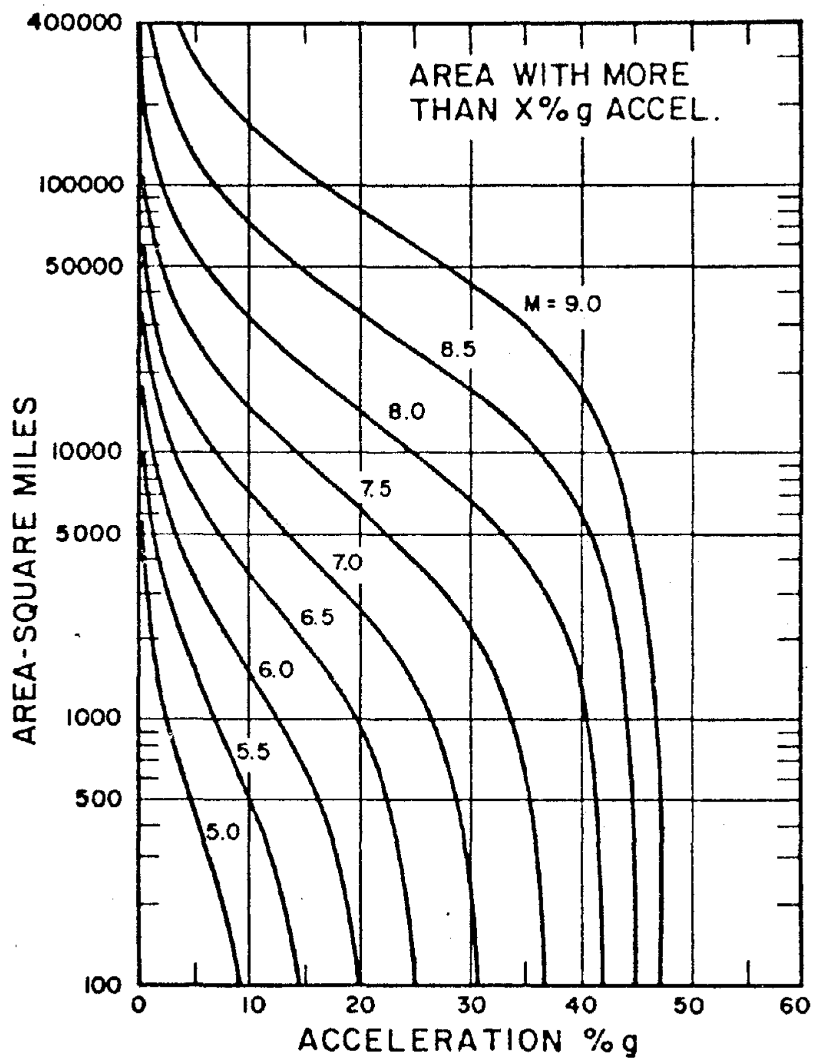


FIGURE 19. AFFECTED AREA CURVES [37]

GROUND ACCELERATION (%g)	MAGNITUDE, M							
	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5
5 - 10	0.4	1.0	2.0	3.2	5.4	14	24	58
10 - 15		0.6	1.0	1.6	3.2	4.4	11	25
15 - 20			0.6	1.1	1.9	3.6	7	14
20 - 25				0.9	1.2	2.0	4	9
25 - 30					1.05	2.0	3.6	7
30 - 35					0.25	1.4	2.4	5
35 - 40						0.6	2.8	6.2
40 - 45							1.2	5.6
45 - 50								0.2

AREA (1000 MI^2)

FIGURE 20. COVERED AREA TABLE

GROUND ACCELERATION	MAGNITUDE, M									
	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5	TOTAL	MEAN
0.05 - 0.10 g	2.12(-2)	1.87(-2)	1.32(-2)	7.45(-3)	4.44(-3)	4.06(-3)	2.45(-3)	2.09(-3)	7.36(-2)	1.47
0.10 - 0.15		1.12(-2)	6.60(-3)	3.72(-3)	2.63(-3)	1.28(-3)	1.12(-3)	9.02(-4)	2.75(-2)	5.50(-1)
0.15 - 0.20			3.96(-3)	2.56(-3)	1.56(-3)	1.04(-3)	7.16(-4)	5.05(-4)	1.03(-2)	2.06(-1)
0.20 - 0.25				2.09(-3)	9.86(-4)	5.80(-4)	4.09(-4)	3.25(-4)	4.39(-3)	8.78(-2)
0.25 - 0.30					8.62(-4)	5.80(-4)	3.68(-4)	2.53(-4)	2.06(-3)	4.12(-2)
0.30 - 0.35					2.05(-4)	4.06(-4)	2.45(-4)	1.80(-4)	1.04(-3)	2.08(-2)
0.35 - 0.40						1.74(-4)	2.86(-4)	2.24(-4)	6.84(-4)	1.37(-2)
0.40 - 0.45							1.23(-4)	2.02(-4)	3.25(-4)	6.50(-3)
0.45 - 0.50								7.22(-6)	7.22(-6)	1.44(-4)

NOTE: NUMBERS IN PARENTHESIS ARE POWERS OF 10

FIGURE 21. ANNUAL NUMBER OF EXPECTED EARTHQUAKES

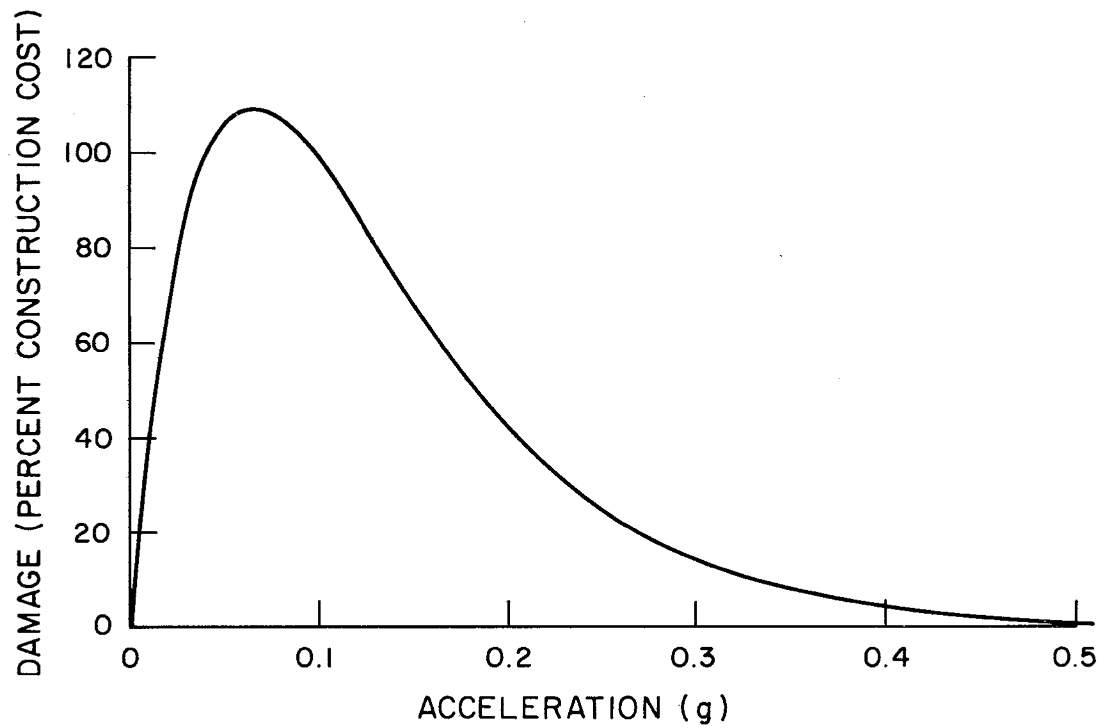


FIGURE 22. LIFETIME DAMAGE PROFILE FOR ONE STORY FRAME

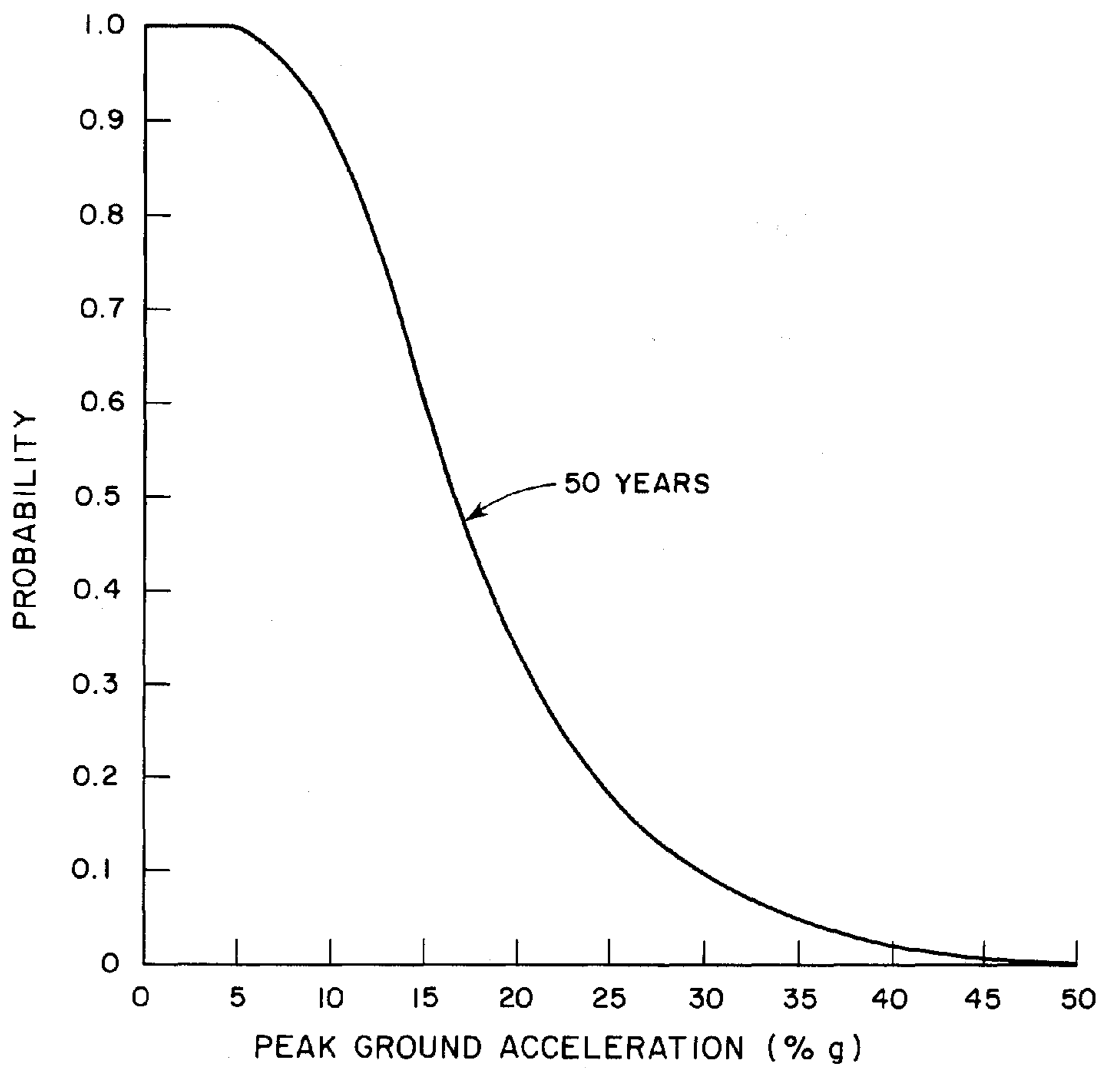
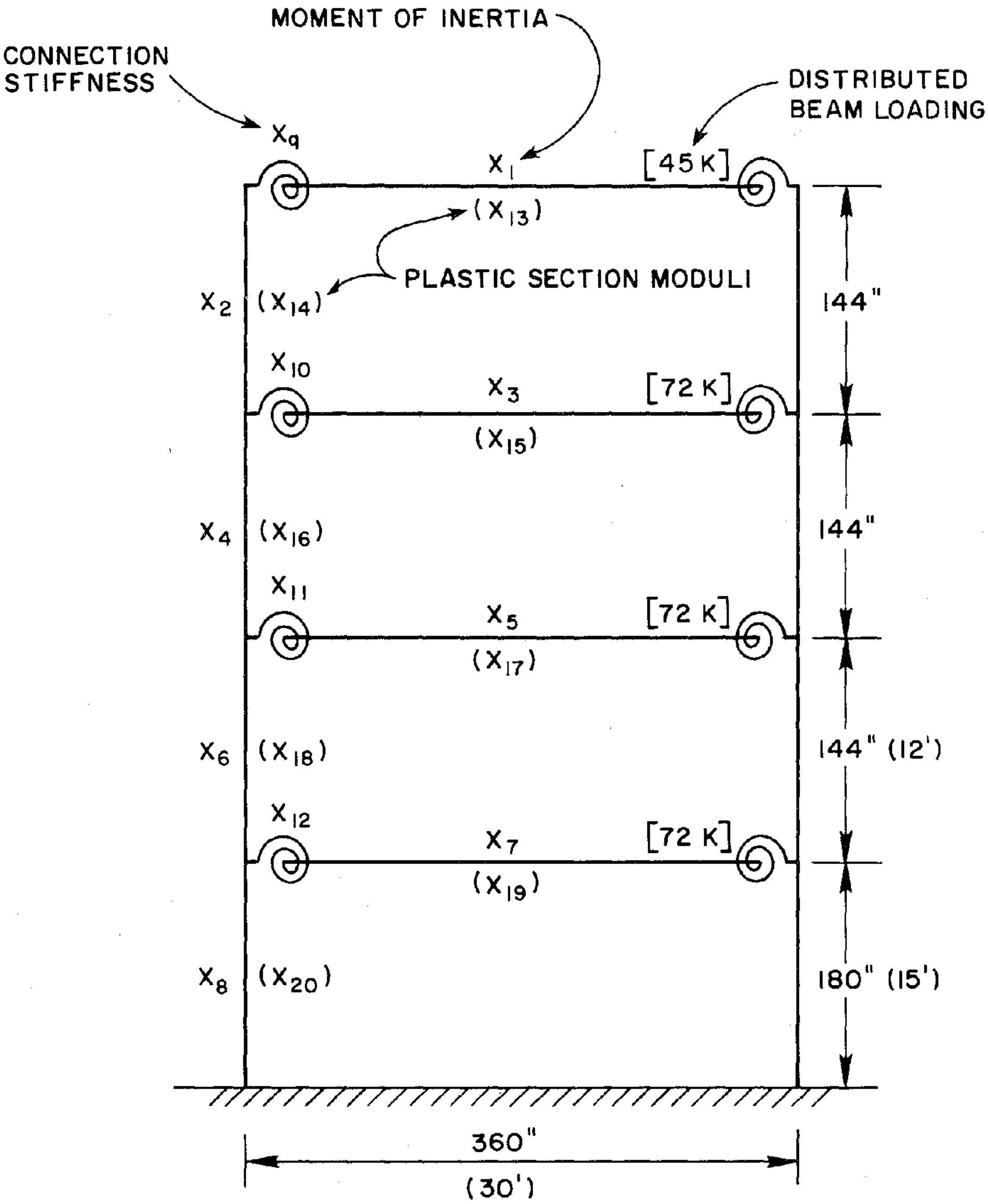


FIGURE 23. PROBABILITY OF OCCURRENCE OF PEAK GROUND ACCELERATION



FRAME SPACING: 288" (24')

FIGURE 24. FOUR STORY EXAMPLE

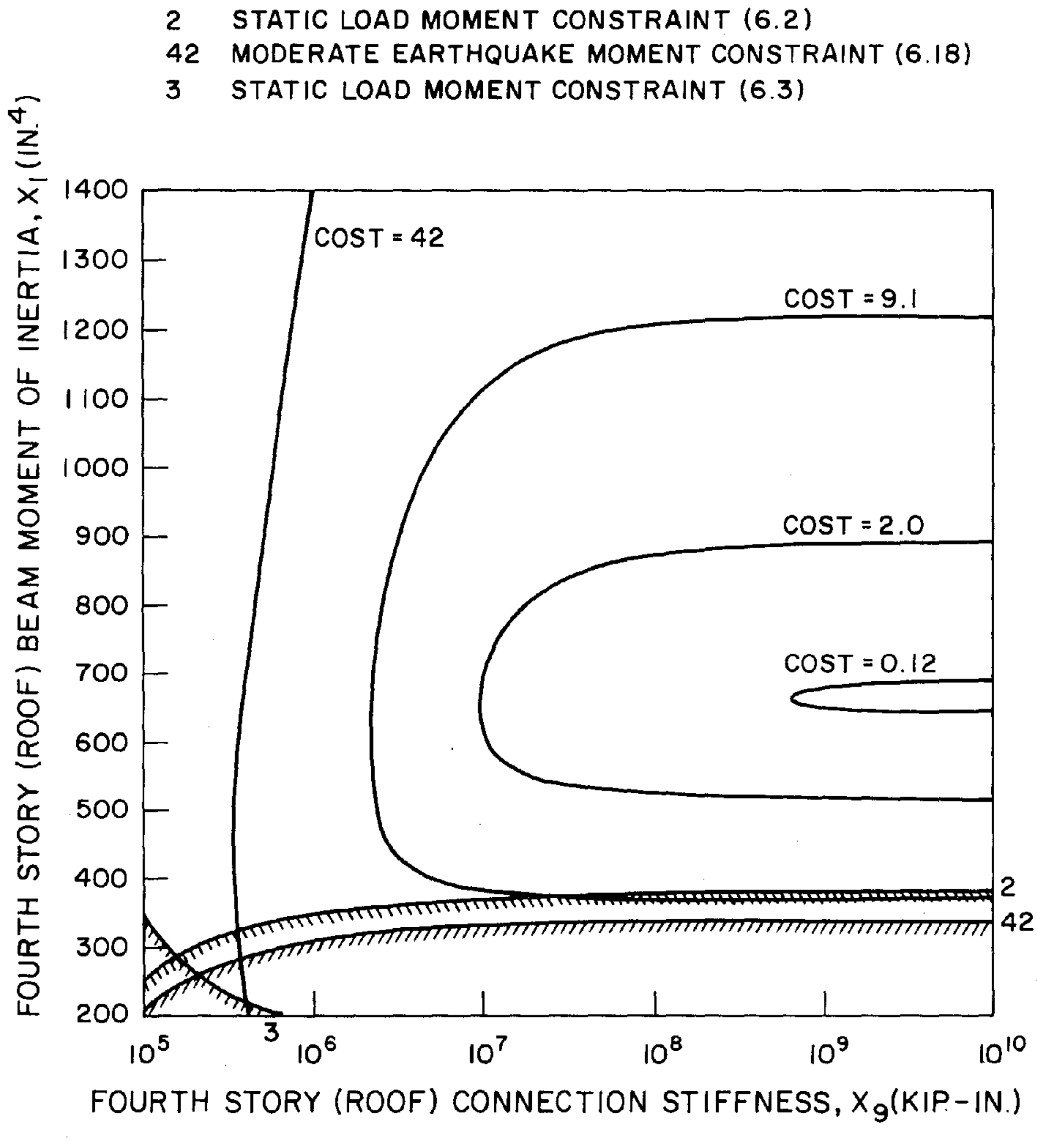


FIGURE 25. BEAM VERSUS CONNECTION DESIGN SPACE

81 STRONG EARTHQUAKE CONSTRAINT (6.22)
83 STRONG EARTHQUAKE CONSTRAINT (6.22)
57 MODERATE EARTHQUAKE CONSTRAINT (6.22)
59 MODERATE EARTHQUAKE CONSTRAINT (6.22)
82 STRONG EARTHQUAKE CONSTRAINT (6.25)
84 STRONG EARTHQUAKE CONSTRAINT (6.25)

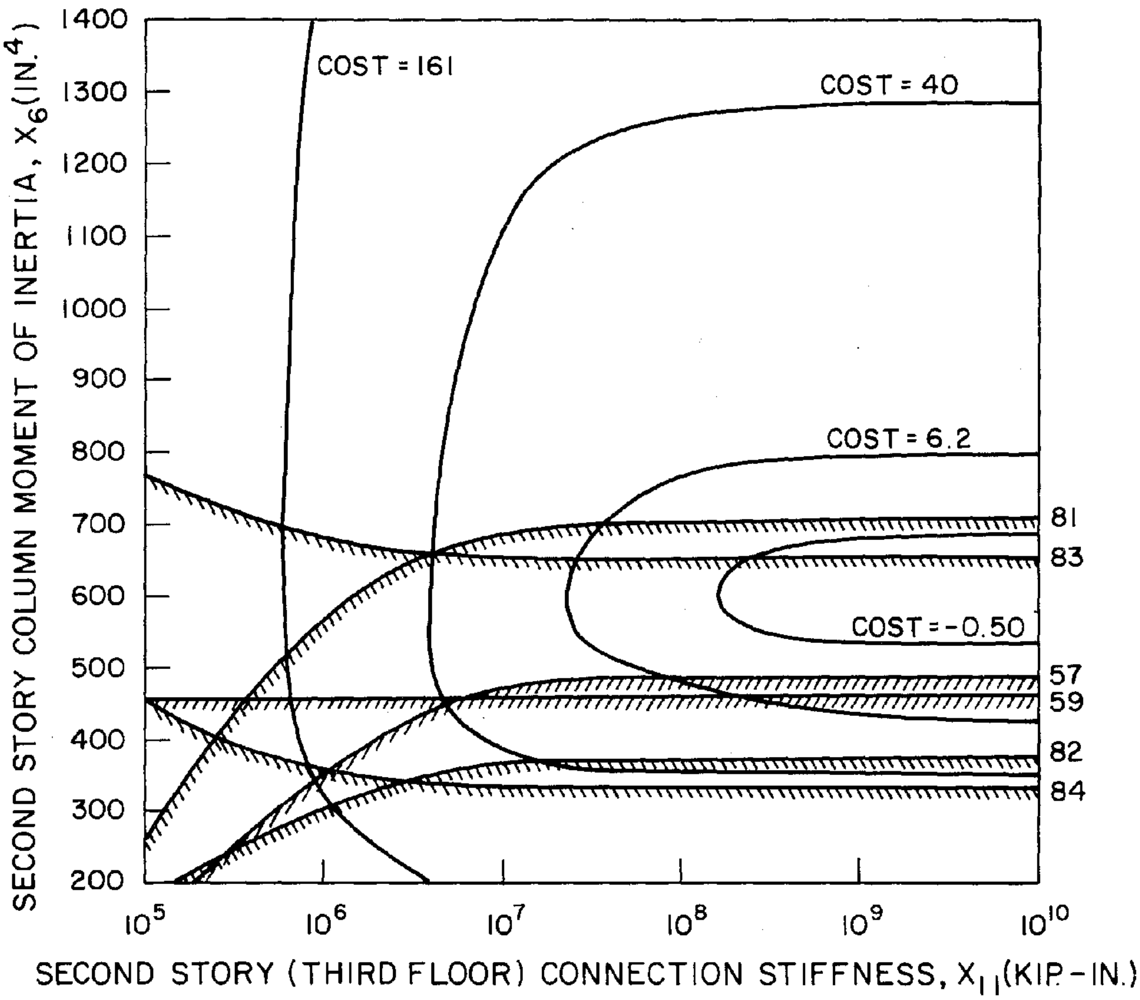


FIGURE 26. COLUMN VERSUS CONNECTION DESIGN SPACE

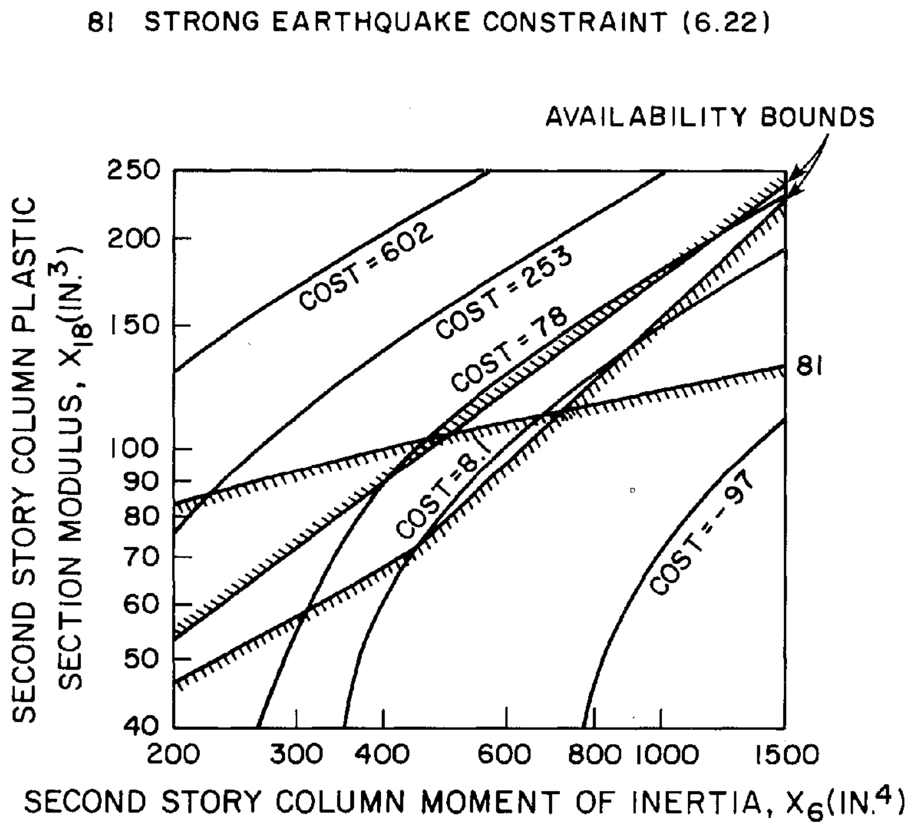


FIGURE 27. COLUMN VARIABLES DESIGN SPACE

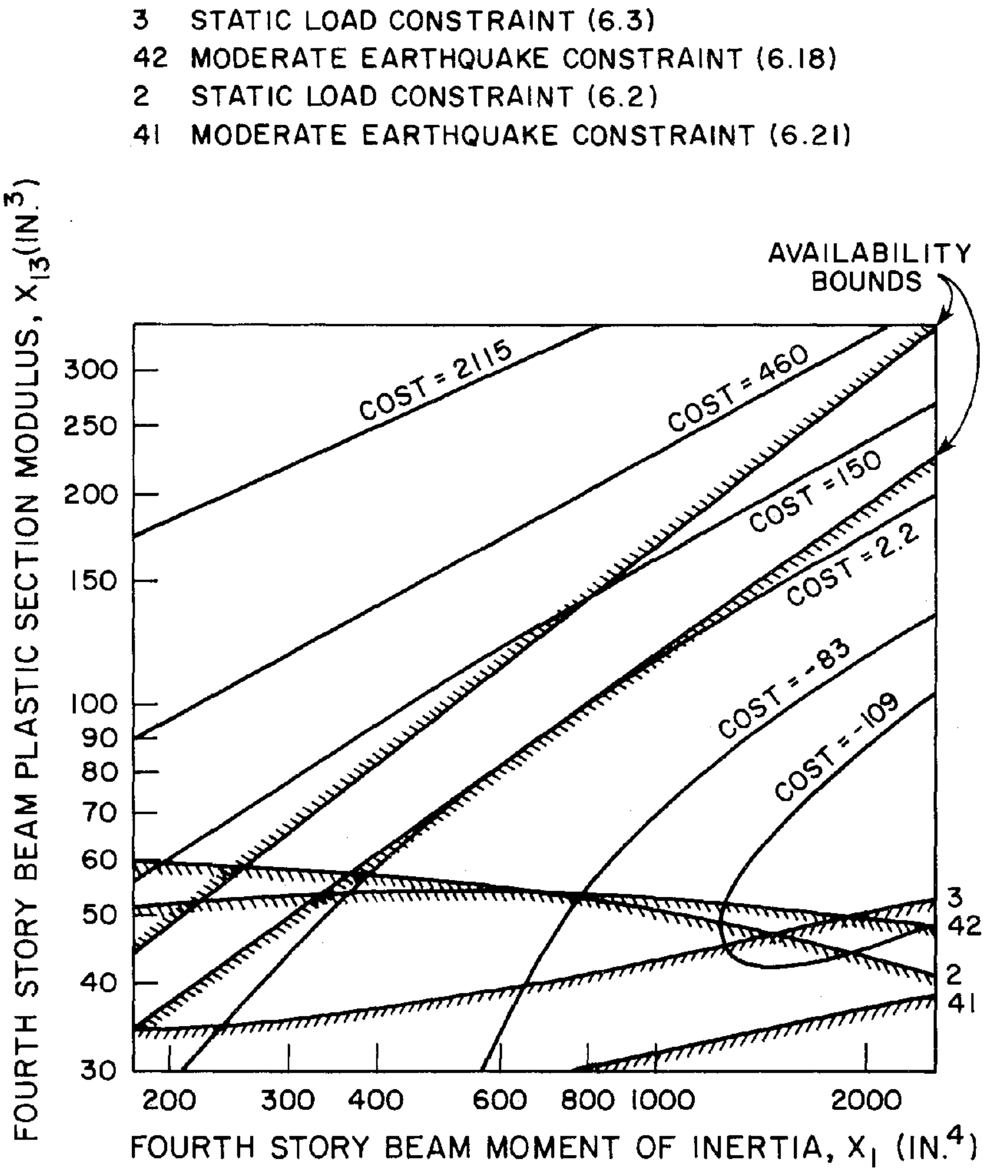


FIGURE 28. BEAM VARIABLES DESIGN SPACE

10 STATIC LOAD MOMENT CONSTRAINT (6.2)
11 STATIC LOAD MOMENT CONSTRAINT (6.3)
45 MODERATE EARTHQUAKE CONSTRAINT (6.21)
46 MODERATE EARTHQUAKE CONSTRAINT (6.18)
53 MODERATE EARTHQUAKE CONSTRAINT (6.22)
77 STRONG EARTHQUAKE CONSTRAINT (6.22)
79 STRONG EARTHQUAKE CONSTRAINT (6.22)

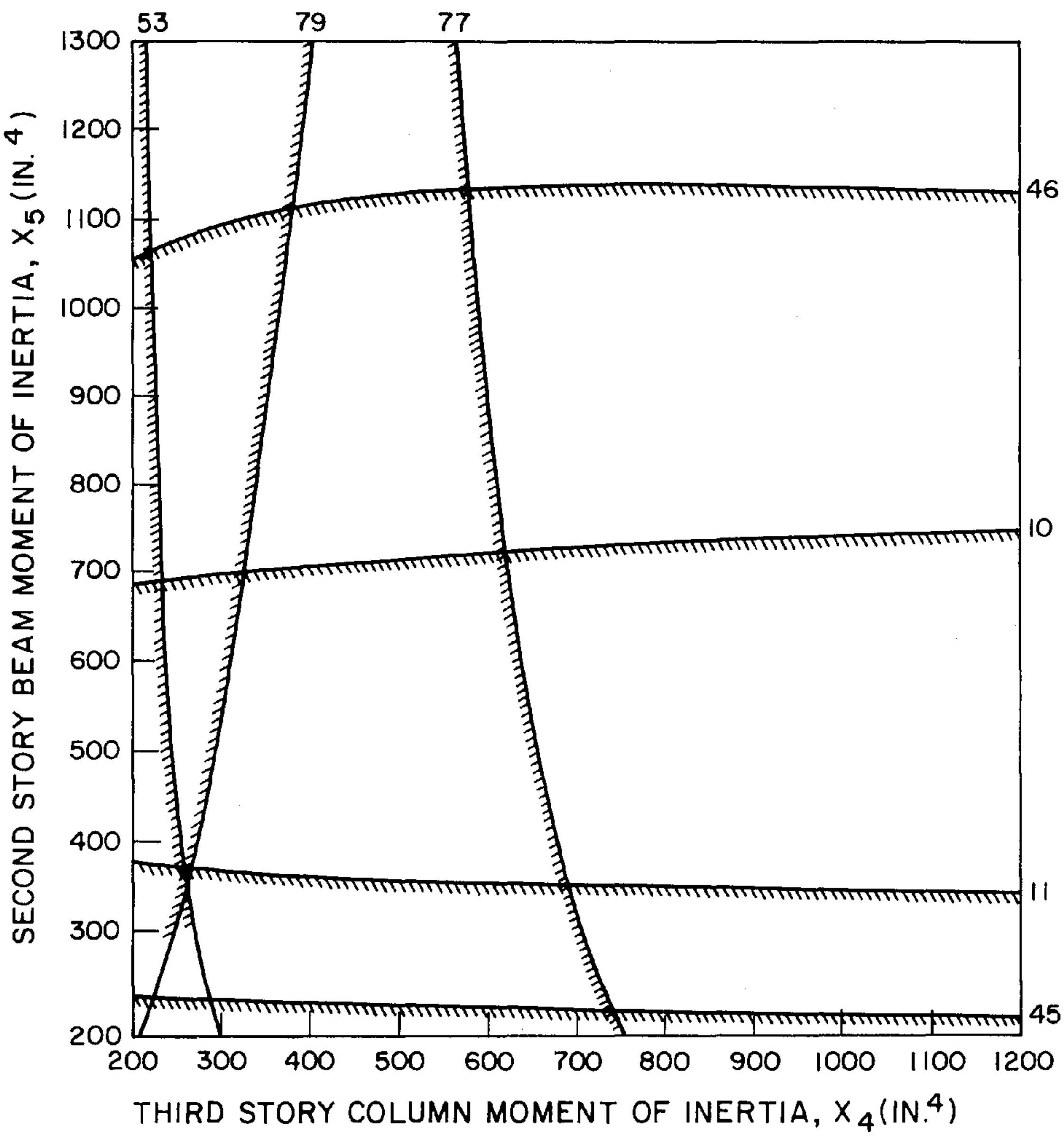


FIGURE 29. BEAM VERSUS COLUMN DESIGN SPACE-CONSTRAINTS

77 STRONG EARTHQUAKE CONSTRAINT (6.22)

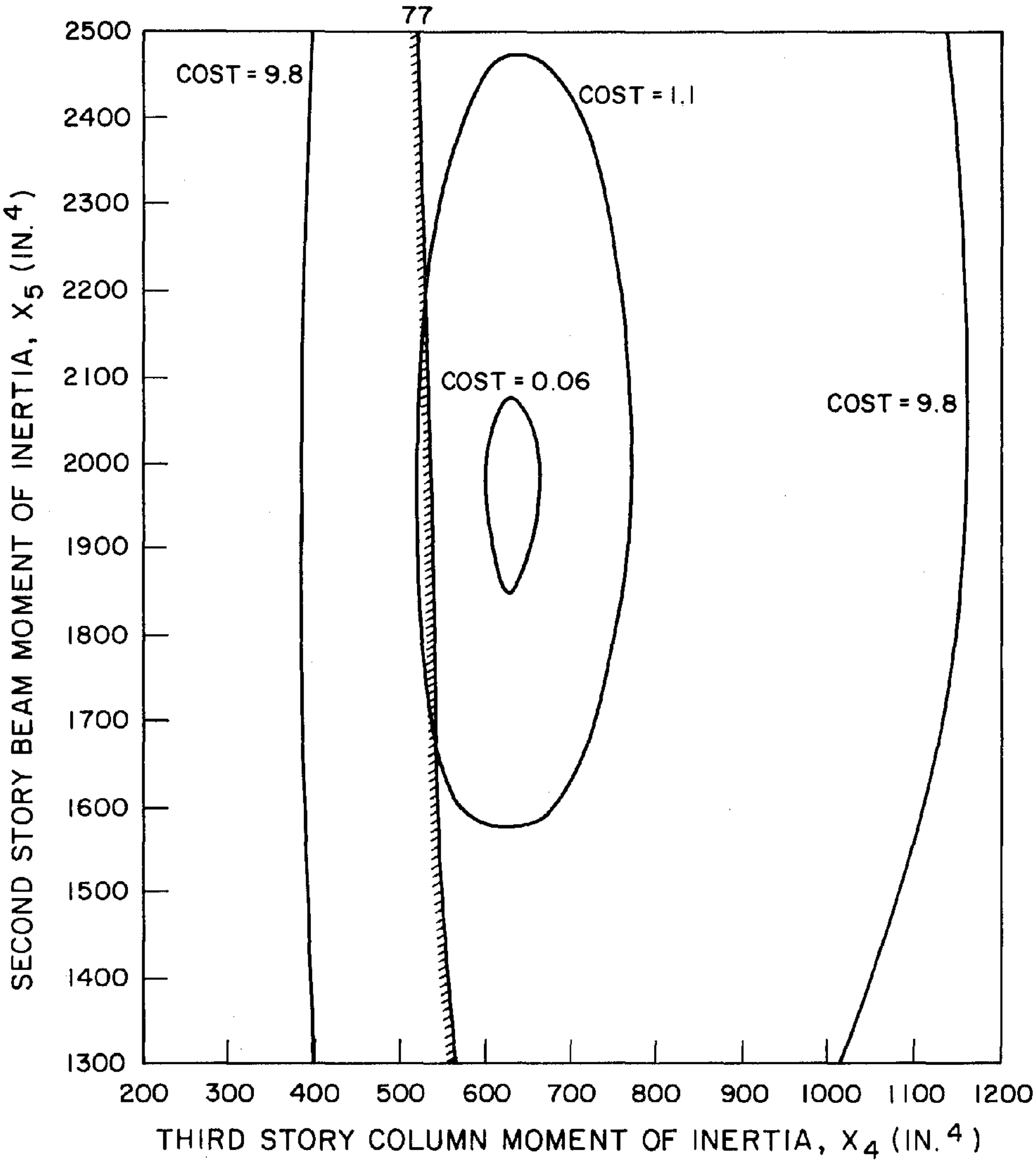


FIGURE 30. BEAM VERSUS COLUMN DESIGN SPACE-COST

53 MODERATE EARTHQUAKE CONSTRAINT (6.22)
77 STRONG EARTHQUAKE CONSTRAINT (6.22)
78 STRONG EARTHQUAKE CONSTRAINT (6.25)
79 STRONG EARTHQUAKE CONSTRAINT (6.22)

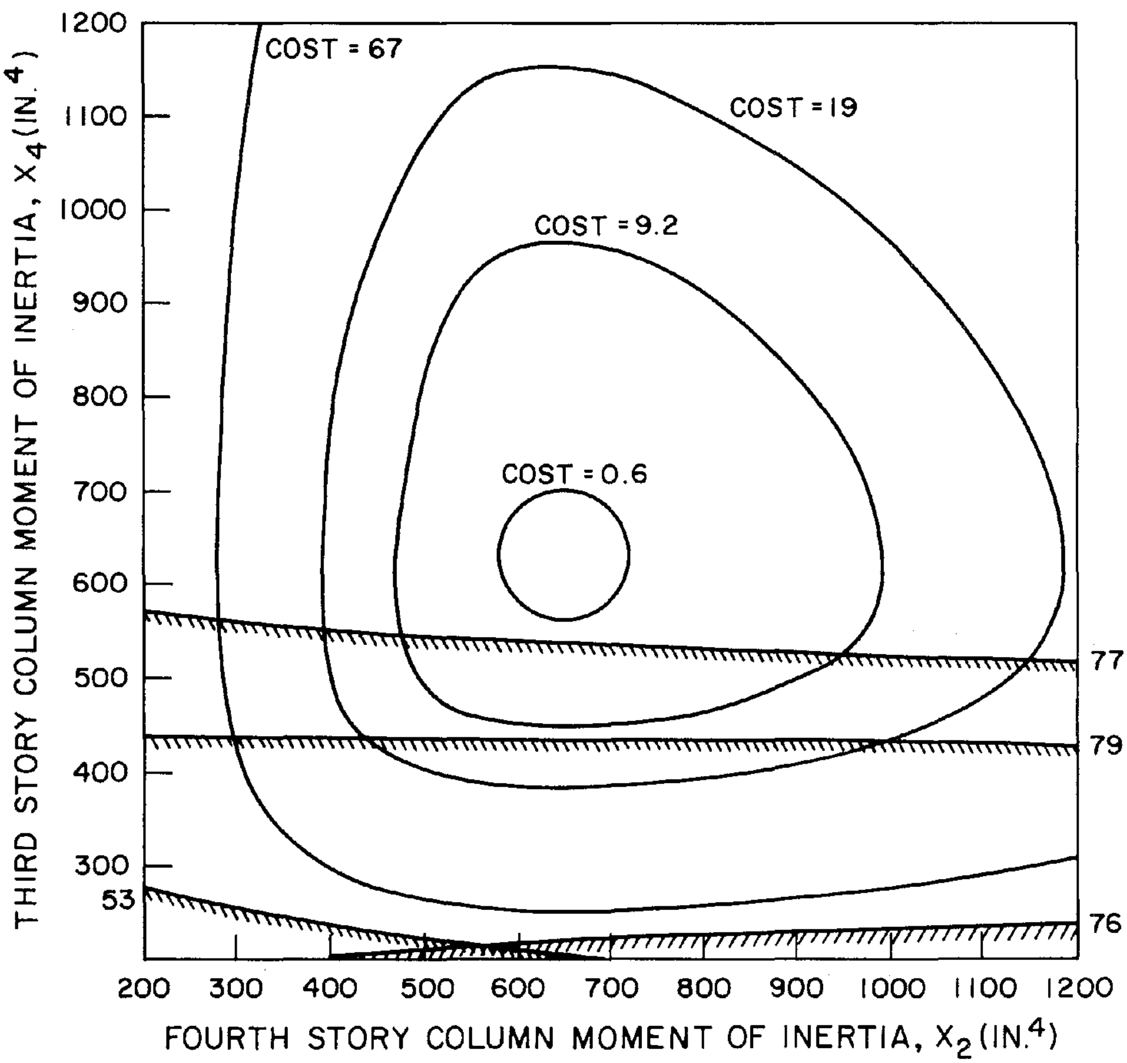


FIGURE 31. COLUMN VERSUS COLUMN DESIGN SPACE

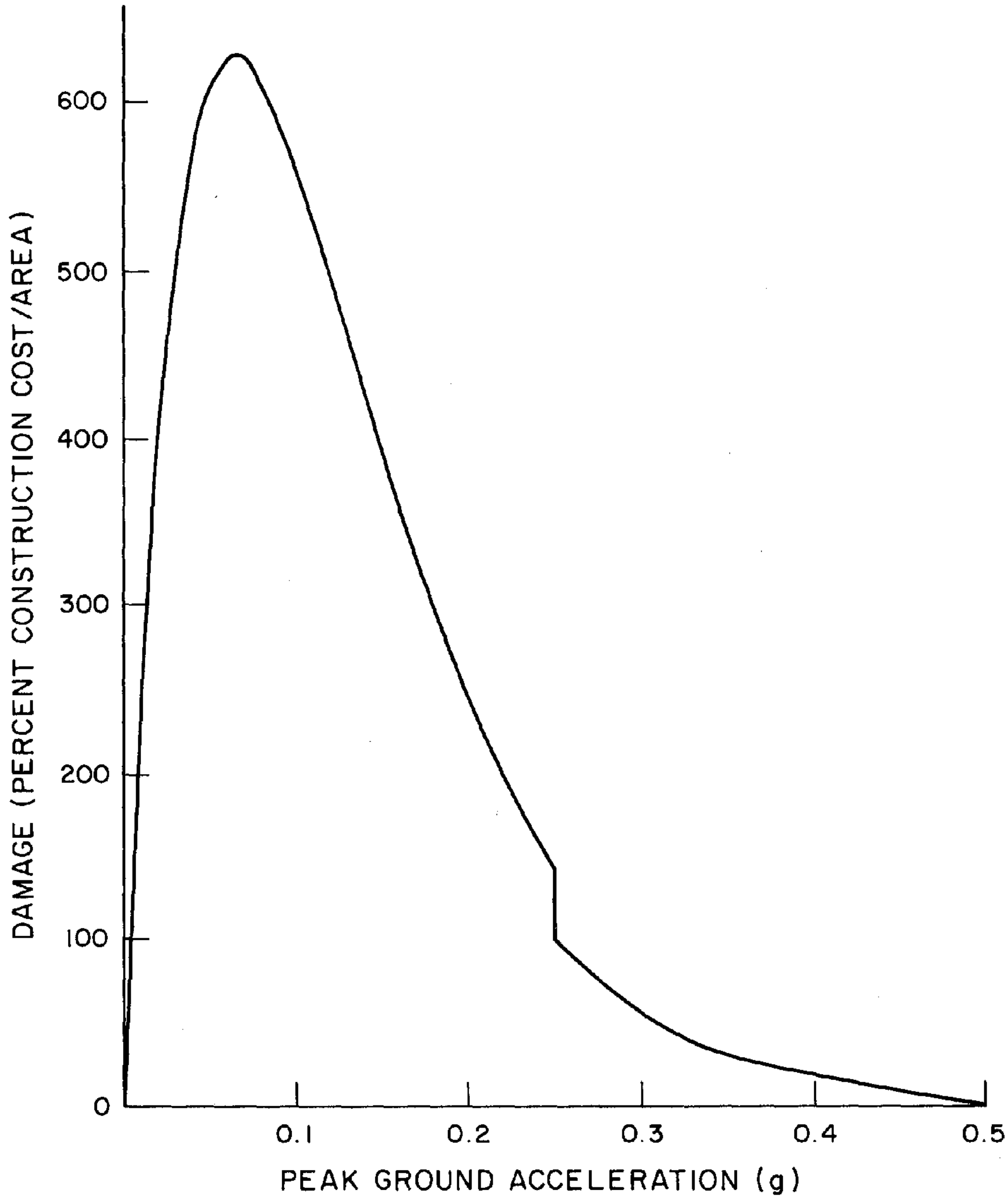


FIGURE 32. LIFETIME DAMAGE PROFILE FOR FOUR STORY FRAME

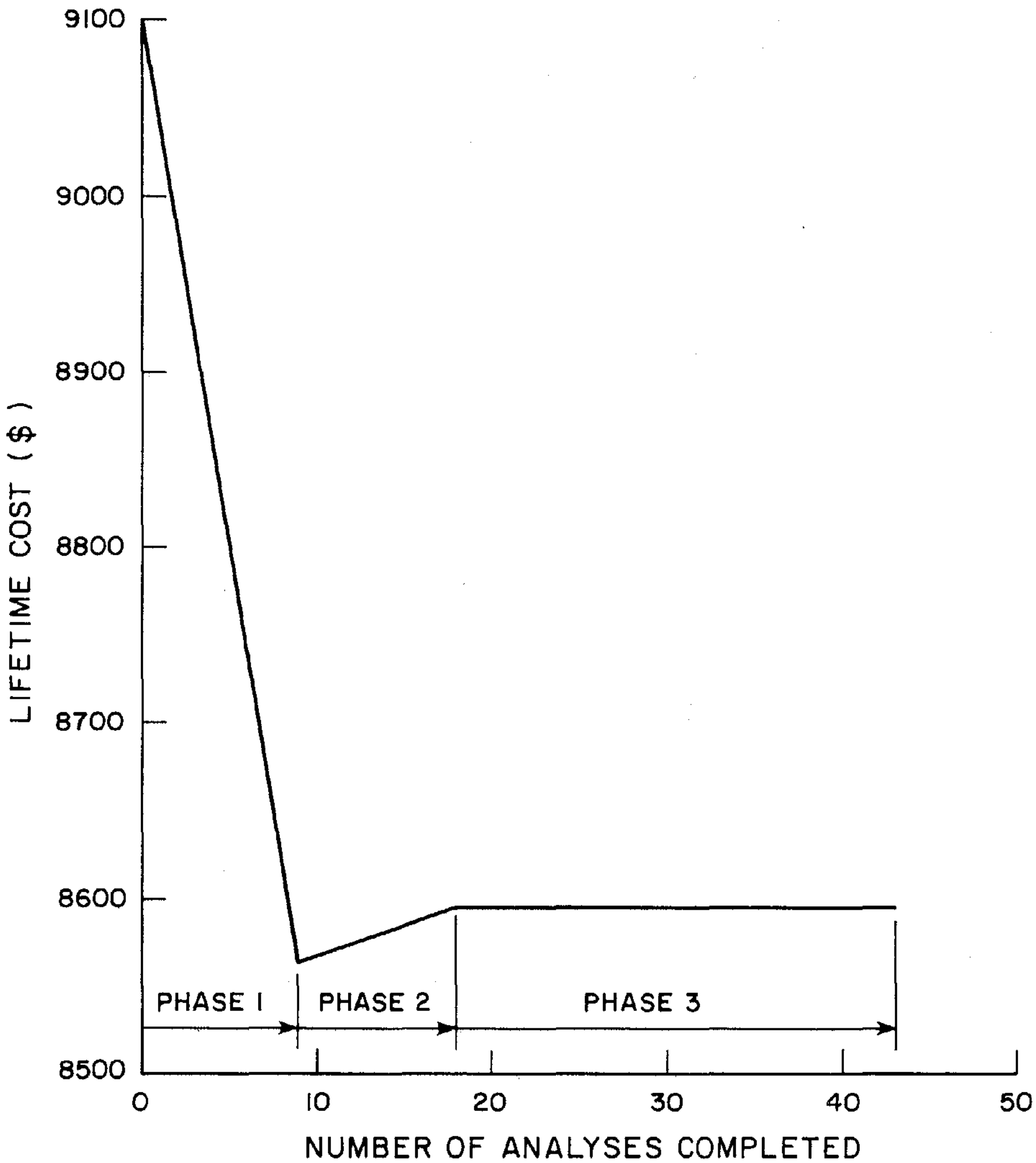


FIGURE 33. ALGORITHM PERFORMANCE

SYSTEM PARAMETER (UNITS)	RESPONSE TO 10% INCREASE IN SYSTEM PARAMETER: OPTIMAL DESIGN CHANGE BEAMS (IN^4)	OPTIMAL DESIGN CHANGE COLUMNS (IN^4)	LC INCREASE (%)	LC INCREASE AT ORIGINAL OPTIMAL (%)	SENSITIVITY ($LC/UNIT SYSTEM PARAMETER)
MEMBER STEEL COST ($/CWT)	81	48	3.92	3.97	135
SIZE-EXTRA CHARGE ($/CWT)	3.1	1.8	0.151	0.153	135
CONNECTION COST (%)	~28	~1.2	0.35	0.35	1.55*
PAINTING COST ($/FT^2)	~7.9	~0.4	0.32	0.32	1435
DESIGN LIFE (YEAR)	170	53	5.03	5.17	45
DAMAGE CURVE SLOPE					
SITE SEISMICITY (%)	170	53	5.03	5.17	22.4*
FLOOR VALUE					
BEAM LOADING (%)	91	26	2.64	--	11.8*
ACTIVE CONSTRAINT (KIP-IN)	--	--	--	--	-0.01

* THESE VALUES DO NOT REPRESENT TRUE SENSITIVITIES AS THEY ARE PER % INCREASE

NOTE: APPROXIMATE VALUES DETERMINED FROM MODEL EXCLUSION EXPERIMENTS

FIGURE 34. SYSTEM SENSITIVITY

APPENDIX

AUTOMATED DESIGN CODE

In this appendix an automated design algorithm, developed on the basis of the preceding text, is presented. This algorithm is essentially a research tool. While some user orientation has been incorporated, the code is far from foolproof. Hence, some familiarity with the theoretical and operational details is necessary for successful operation. With this in mind a general discussion of the algorithm, its input and output follows.

The code is composed of two parts: (i) an automated design portion and (ii) a structural analysis portion. The routines associated with each are as follows:

Automated Design	Structural Analysis
DESIGN	NERD
AM	F
PROJ	ROOT
EXTP	ANAL
INTP	EIGEND
	MATMUL
	NORMV
	FDRIV
	DERIV
	HQRW

In the automated design portion of the code, DESIGN is the main routine and embodies the four-phase algorithm given in section 8. Subroutine AM stores the A matrix; PROJ generates the projection operator; EXTP computes a Newton step and INTP a derivative estimated formula step for DESIGN as needed.

For the structural analysis part of the computer program, NERD serves as the driver routine and assimilates all necessary structural data. Subroutine F is a function generator wherein the cost function and all constraint functions are computed. The essential raw data for F is generated in ROOT, which is a coded solver for (6.11), and in ANAL which is the primary structural analysis routine. In ANAL the mass and stiffness matrices are formulated and the static and dynamic problems solved. Subroutines EIGEND, MATMUL, NORMV and HQRW are service routines for ANAL. Subroutine FDRIV is the function gradient generator for the cost and constraint functions with DERIV providing raw data to FDRIV.

The primary program direction comes from DESIGN with F and FDRIV called upon as needed in the design process. Most of the computational time is spent in F and FDRIV, however. Thus, some attempt at making these routines efficient has been made (see [38]). To a considerable degree the design and the analysis portions of the code are autonomous. Hence, it would be fairly easy to attach new automated design apparatus to the present analysis portion and vice versa. Thus, the two parts of this code have use beyond their present setting.

At present the design part of the code is capable of formulating only single bay multistory frames with fixed connection stiffnesses, any value of connection stiffness may be used, however. In addition member option sets are restricted to AISC economy sections. The analysis portion of the computer program is somewhat more general than as presently utilized by the design algorithm. It is capable of handling any set of connection stiffness values and any combination of moment of inertia and plastic section modulus quantities. It could thus be used in a more general design automation scheme.

The algorithm is written in Fortran IV. It operates completely within core and requires 46362 words of central core storage to load and execute. The program was written for and has been operated on the CDC 6400 located at the University of California at Berkeley.

With this background then the input for the code is as follows:

Card 1. NSTR [I5]

NSTR is the number of stories. Present dimension statements limit the number of stories to eight or less.

Card 2. EL(NSTR) [8F10.0]

EL(•) are the story heights (column lengths) in inches, starting from the top story and listed down to the first story.

Card 3. ELB [F10.0]

ELB is the bay width (beam length) in inches.

Card 4. B(5 x NSTR) [8F10.0]

B(•) are the components of the initial design vector. Member moments of inertia (in^4) are listed first starting with the top member in the structure (roof beam) and working down to the bottom member (first story column). The connection stiffnesses (kip-in) are listed second, again from the top down. For fully rigid connections stiffnesses of 9×10^9 or 10^{10} have been used herein. Listed last are the member plastic section moduli (in^3) arranged in the same fashion as the moments of inertia. For an example listing see figure 24 wherein the plastic section moduli are in parentheses. Once the program starts, appropriate plastic section moduli are automatically computed for the changing design vector (see card 13). For the initial design however they must be specified according to the lower bounds (3.3) and (3.4). Note that more than one card and as many as five may be required to specify the initial design vector.

Card 5. W(NSTR) [8F10.0]

W(•) are the distributed dead/live load on the beams. These values represent the total distributed load in kips on each beam and are to be listed starting with the roof beam and proceeding on down to the first story beam.

Card 6. DEQ(2) [2F10.0]

DEQ(•) are the design earthquakes. The moderate earthquake is listed first and the strong earthquake second. The values are to be specified in peak ground acceleration divided by g.

Card 7. CSM, CSC, CTS, CSW, CPT, COH, PFT [7F10.0]

CSM represents the cost of steel for the structural members in dollars per cwt.

CSC signifies the cost of steel for the connection plates in dollars per cwt.

CTS is the cost of transporting steel in dollars per cwt.

CSW specifies the cost of welding in dollars per cwt of weld metal deposited.

CPT is the cost of painting in dollars per square inch.

COH represents the cost of overhead divided by construction cost.

PFT specifies profit divided by construction cost.

Card 8. DTCM, DL, NAG [2F10.0,I5]

DTCM signifies the down time cost divided by total damage cost. This input variable can also be used to enter an estimated building construction cost if desired. In this case the negative of the building construction cost times one plus DTCM is input here instead of DTCM. If a design automation sequence is being restarted this option must be employed in order to avoid solving a different problem (see card 10).

DL is the design life in years.

NAG specifies the number of integration points to be used in the trapezoidal estimation of the area under the lifetime damage profile (lifetime damage estimate). The maximum number allowed is 10.

Card 9. AG(NAG) [8F10.0]

AG(·) are the integration points (peak ground acceleration divided by g) to be used in the trapezoidal estimation of the lifetime damage. This list must begin with 0, terminate with 0.5 and progress sequentially through the desired integration points.

Card 10. FP(NSTR) [8F10.0]

FP(·) specify the floor participation values which are to be input from the top floor down to the first floor. These quantities represent the cost of glass and drywall partitions for a particular floor divided by the building construction cost. In order to use these values to compute nonstructural damage an estimated building construction cost is needed. This value can either be input through DTCM as noted above or the program can be allowed to compute one. In the latter situation this estimate is based on the initial design vector (card 4); the costs of card 7 and the assumption that the steel frame constitutes 12.5% of the building construction cost [33]. This building cost estimate is retained throughout the design process in order to keep the floor values constant. Thus, if this set of data represents a design sequence restart, the original building cost must be input through DTCM in order to be solving the same problem.

Card 11. NCC, NSC [2I5]

NCC is the number of components in the design vector which are to be constrained to their original (or internally determined) value. Up to 38 components may be fixed. The way DESIGN is presently set up NCC must include all the connection stiffnesses and member plastic section moduli. Thus, NCC must at the minimum equal 3 x NSTR.

NSC is the number of components of the design vector which are to be internally computed. NSC must be included in NCC, hence, NCC $\geq$ NSC. The only components of the design vector eligible for internal computation are the member plastic section moduli. If this option is used, then the selected plastic section moduli will be computed on the basis of the lower bounds of equations (3.3) and (3.4) (economy sections). As DESIGN is presently set up all the member plastic section moduli must be included here, thus, NSC = 2 x NSTR.

Card 12. MC(NCC) [16I5]

MC(·) signify the components of the design vector which are to be constrained. These must be listed in increasing numeric order. For example if x_2, x_3 and x_9 are to be confined then 2, 3 and 9 should be entered here in that order. These cards are necessary only if NCC > 0, otherwise there is no card 12.

Card 13. MS(NSC) [16I5]

MS(·) are the members for which the plastic section moduli are to be determined internally. The member label to use here is the moment of inertia. For example in figure 24 if x_{16} is to be computed according to the lower bound of (3.4) then the number 4 should be entered here since it corresponds to the associated moment of inertia. These numbers should be listed in increasing numeric order. These cards are required only if NSC > 0.

Card 14. COP, CC(2), CE(2) [5F10.0]

This card is used to input the mathematical description of the economy sections for the columns in terms of power functions. At present this description is given by (3.4) so that COP = 429, CC(1) = 2.47, CC(2) = 0.21, CE(1) = 0.553 and CE(2) = 0.956. If a more appropriate description is needed for a more limited or perhaps enlarged option set, then this new model can be input here. This card is necessary only if NSC > 0.

Card 15. <u>RCS, RCD, BFS</u> [3F10.0]

RCS is the reduction coefficient for the static moment constraints (c in equations (6.1) - (6.7)).

RCD is the reduction coefficient for the moderate earthquake moment constraints (c in equations (6.13) - (6.25)).

BFS is the buckling safety coefficient (c in equation (6.10)).

Card 16. <u>DA(NSTR)</u> [8F10.0]

DA(·) are the mid-beam displacement allowables listed from the top story (roof beam) down. These allowables are for the dead/ live loads given in card 5. If allowables for only live loads are desired, then the live load allowable should be scaled up according to

$$\text{DA}(\cdot) = (\text{Live load allowable}) \times \frac{(\text{Dead load}) + (\text{Live load})}{(\text{Live load})}$$

This is admittedly approximate, but it is good enough for this application.

Card 17. <u>DUCA(2 x NSTR)</u> [8F10.0]

DUCA(·) represent the ductility demand allowables for the strong earthquake design limits (see section 6.2). These values must be entered for each member from the roof beam down to the first story column. (Listed in identical order to the moments of inertia.)

Card 18. <u>BSUL, BSLL, BSM, CSUL, CSLL, CSM</u> [6E10.0]

In order to use the method of bisection, in each coordinate (design vector component) direction an interval which contains the minimum must first be established. This is done automatically within the code on the basis of the information submitted on this card. It is accomplished by searching along each coordinate direction in constant steps until the requisite intervals are constructed. The step size used in each coordinate search is directly proportional to the associated directional derivative. The constant of proportionality for the beams is BSM and for the columns CSM. In order to insure step sizes which are neither too large nor too small, bounds are imposed on them. The upper bounds are BSUL and CSUL for the beam and column step sizes respectively and the lower bounds are BSLL and CSLL. Suggested values for these parameters are BSM = $5(10^3)$, CSM = $3(10^3)$, BSUL = 500, BSLL = 2, CSUL = 300, and CSLL = 2. These are simply suggested values. Experimentation is warranted with each new problem type, particularly with regard to BSM and CSM.

Card 19. BAC, EPSM, EPSD, BHL, CHL, RHO [6E10.0]

BAC represents the interval length for the termination of the method of bisection. The suggested value is 20.

EPSM is the numerical tolerance allowed in moment constraint function location determination. The suggested value is 50. This value is divided by 10 for the final optimal design specification.

EPSD is the numerical tolerance allowed in beam displacement constraint function location finding. This value is divided by 10 for the final optimal design search. The suggested value is 0.5.

BHL specifies the optimal design search termination value for the directional derivative of beam-related components. This value is an indirect specification of η in the gradient projection algorithm of section 8. The value used herein was $1(10^{-3})$.

CHL specifies the optimal design search termination value for the directional derivative of column-related components. This value is an indirect specification of η in the gradient projection algorithm of section 8. The value used herein was $3(10^{-3})$.

RHO represents the initial step size for the gradient projection algorithm of section 8. This value is adjusted internally according to how the problem is progressing and thus does not remain constant. A suggested value is 20.

Card 20. KCL, KNL, KL, NGL [4I5]

KCL signifies the maximum number of iterations allowed in the search for the unconstrained minimum. This number includes both the interval establishment apparatus mentioned above and the bisection procedure. The suggested limit is 20.

KNL specifies the number of phase 1 restarts desired. Since the cost function is not truly uncoupled it may be beneficial in some cases to pass through phase 1 more than once. If so, the desired number of passes is specified by KNL. In this study more than one pass was rarely found to be beneficial. Indeed, because the phase 1 apparatus is relatively crude additional passes sometimes made the situation worse rather than better. The suggested value is 1.

KL is the maximum number of iterations allowed in searching for constraint functions. The suggested value is 5.

NGL represents the maximum number of gradient projection iterations allowed in phase 3. This value obviously depends on the termination levels specified by BHL and CHL and thus a hard rule is not possible in this regard. This parameter is specified only to protect against a runaway code as is KCL and KL, hence, some judgement is necessary. The limit used herein was 30.

Card 21. KPC(·) [16I5]

KPC(·) represent the primary constraints. A maximum of 37 primary constraints may be specified. These values must be listed in increasing numeric order and be terminated by a blank input field (KPC(·) = 0). For an outline of how constraints are labeled see the output discussion which immediately follows the input discussion.

Card 22. K [I5]

The algorithm can be started at phase 1 or phase 2 and can be instructed to skip part of phase 2 for essentially what amounts to a phase 3 start. For a regular (phase 1) start a blank card (K = 0) or K = 1 should be specified. A phase 2 start is initiated by stipulating K = 2. With a phase 2 start all the phase 1 apparatus is skipped. Note, when a phase 2 start is called for the usable design which results from phase 2 is adjusted to approximate the projection of the unconstrained optimal (specified via BU(·) in the following card) onto the surface of active constraints. If simply a usable design is desired from phase 2 without this adjustment then K = 3 should be input, the adjustment apparatus at the close of phase 2 will then be skipped as well as the phase 1 procedures. When the initial design vector submitted via card 4 is usable, inputting a K of 3 amounts to a phase 3 start. The algorithm will not proceed to phase 3, however, regardless of what is called for without first checking for and generating, if necessary, a usable design. With K and the parameters of cards 19, 20 and 21 the algorithm can be used to do almost anything the user may desire, a little ingenuity being all that is necessary.

Card 23. BU((5 x NSTR) - NCC) [8E10.0]

When $K > 1$, phase 1 of the algorithm is skipped. In this case the unconstrained optimal must be supplied by the user. Only those components of the design vector which are not confined have to be supplied. They must be submitted in the same order as specified in card 4 with the constrained components eliminated. These cards are not necessary for a normal (phase 1) start.

The output from the algorithm is well-labeled and fairly self-explanatory. It begins with a print out of the input data to facilitate checking. Periodically throughout the proceedings the total number of structural analyses which have been conducted is printed out. The presiding value of the cost function is also frequently noted. At the close of the algorithm the complete optimal design vector is printed and all active constraints at the optimal are listed. In addition the values of all the constraint functions at the optimal are produced. In this listing the constraints are labeled consecutively from 1 to 22 x NSTR. The listing begins with the static load beam moment-constraints (6.1), (6.2), (6.3) and (6.4), in that order for each beam from the roof beam down to the first-story beam. This complete set is followed in the listing by the static load column moment-constraints (6.6) and (6.7), first for the top end-moment and then the bottom end-moment, beginning with the top-story column and proceeding on down to the first-story column. After this set the beam displacement constraints (6.9) follow, again from the roof beam down. The sidesway stability limits (6.10) are next from the top-story down. The static loading constraints are followed by the moderate earthquake moment restrictions beginning with the beam constraints (6.21) and (6.18) in that order for each beam top down and concluding with the column constraints (6.22) and (6.25) for the top end-moment and the bottom end-moment for each column from the top-story down. The strong earthquake constraints are listed last and involve the same inequalities and follow the same order as the moderate earthquake restrictions.

In summary then the constraints are labeled as follows:

$$\text{First}\quad \left.\begin{array}{l}(6.1)\\(6.2)\\(6.3)\\(6.4)\end{array}\right\}\ \text{repeated for each beam from the top-story down to the first-story.}$$

This complete set is followed by

(6.6)
(6.7) top end-moment
(6.6)
(6.7) bottom end-moment

repeated for each column top down,

followed by (6.9) for each beam top down,

followed by (6.10) for each column top down,

followed by (6.21)
(6.18) for moderate earthquake loading for each beam top down,

and (6.22)
(6.25) top end-moment
(6.22)
(6.25) bottom end-moment

for moderate earthquake loading for each column top down.

The set of moderate earthquake constraints is then repeated identically for the strong earthquake loading.

A complete listing of the design algorithm follows, including an example two-story structure which illustrates much of the preceding discussion.

The following computer program listing is available upon request from Professor K. Pister, Structural Engineering and Structural Mechanics, Davis Hall, University of California, Berkeley, California.

```
* * * * * * * * * * * * * * * * * * * *
*    AUTOMATED DESIGN PROGRAM LISTING  *
* * * * * * * * * * * * * * * * * * * *

      PROGRAM NERD(INPUT,OUTPUT)
      COMMON /STDAT/NSTR,W(8),ELB,EL(8),Q,NCS2,NCS3,NCS4,NCS5,NCS6,NCSL,
     $NCO
      COMMON /COST/CSMP,CSCP,CSW,CPT,COH,CI,FP(8),NAG,AG(10),DL
      COMMON /CNSTINF/RCS,DA(8),BFS,RCD,DEQ(2),DUCA(16)
      DIMENSION B(40)
      PRINT 5
    5 FORMAT (///5X,*----- NERD INPUT DATA -----*)
      READ 10, NSTR
   10 FORMAT (16I5)
      PRINT 12,NSTR
   12 FORMAT (//2X,*NUMBER OF STORIES---*,I3)
      READ 20,EL,ELB
   20 FORMAT (8F10.0)
      PRINT 22,(EL(I),I=1,NSTR)
   22 FORMAT (/2X,*STORY HEIGHTS (TOP DOWN)---*,8F10.2)
      PRINT 24,ELB
   24 FORMAT (/2X,*BAY WIDTH---*,F10.2)
      NCS2=NSTR+NSTR
      NCS3=NCS2+NSTR
      NCS5=NCS2+NCS3
      READ 20,(B(I),I=1,NCS5)
      PRINT 26,(B(I),I=1,NCS2)
   26 FORMAT (/2X,*MEMBER MOMENTS OF INERTIA (TOP DOWN)---*,8F11.2/40X,8
     $F11.2)
      IS=NCS2+1
      PRINT 28,(B(I),I=IS,NCS3)
   28 FORMAT(/2X,*CONNECTION STIFFNESSES (TOP DOWN)---*,8F11.0)
      IS=NCS3+1
      PRINT 30,(B(I),I=IS,NCS5)
   30 FORMAT (/2X,*MEMBER PLASTIC SECTION MODULI (TOP DOWN)---*,8F9.2/45
     $X,8F9.2)
      READ 20,W
      PRINT 32,(W(I),I=1,NSTR)
   32 FORMAT(/2X,*BEAM LOADING (TOP DOWN)---*,8F10.2)
      READ 20,DEQ
      PRINT 34,DEQ
   34 FORMAT (/2X,*DESIGN EARTHQUAKES---*,2F10.3)
      READ 20,CSM,CSC,CTS,CSW,CPT,COH,PFT
      PRINT 36,CSM,CSC,CTS,CSW,CPT,COH,PFT
   36 FORMAT (//2X,*CONSTRUCTION COSTS:*//5X,*MEMBER STEEL---*,F8.2//5X,
     $*CONNECTION STEEL---*,F8.2//5X,*STEEL TRANSPORTATION---*,F8.2//5X,
     $*WELD METAL---*,F8.2//5X,*STRUCTURAL PAINTING---*,F8.6//5X,*OVERHE
     $AD---*,F8.2//5X,*PROFIT---*,F8.2)
      READ 39,DTCM,DL,NAG
   39 FORMAT (2F10.0,I5)
      IF(DTCM.LT.0.0) GO TO 41
      PRINT 40,DTCM,DL
   40 FORMAT (//2X,*DAMAGE COSTS:*//5X,*DOWN TIME COST MULTIPLIER---*,
     $F7.2//5X,*DESIGN LIFE---*,F8.2)
      CI=-8./(1.+DTCM)
      GO TO 43
   41 CI= DTCM
      PRINT 42,CI,DL
   42 FORMAT (//2X,*DAMAGE COSTS:*//5X,*INITIAL COST TIMES (ONE PLUS DOW
     $N TIME COST MULTIPLIER)---*,F10.2//5X,*DESIGN LIFE---*,F8.2)
   43 READ 20,(AG(I),I=1,NAG)
      PRINT 44,(AG(I),I=1,NAG)
   44 FORMAT (/5X,*COST ESTIMATION EARTHQUAKES---*,10F8.3)
      READ 20,FP
      PRINT 45,(FP(I),I=1,NSTR)
   45 FORMAT (/5X,*FLOOR PARTICIPATION FACTORS (TOP DOWN)---*,8F10.4)
      READ 10,NCC,NSC
      IF(NCC.EQ.0) GO TO 52
      READ 10,(MC(I),I=1,NCC)
      PRINT 46,NCC,(MC(I),I=1,NCC)
   46 FORMAT (//2X,*THERE ARE*,I3,* CONSTRAINED COMPONENTS IN THE DESIGN
     $ VECTOR:*,19I4/59X,18I4/59X,2I4)
      IF(NSC.EQ.0) GO TO 52
      READ 10,(MS(I),I=1,NSC)
      PRINT 48,NSC,(MS(I),I=1,NSC)
   48 FORMAT (/2X,*THERE ARE*,I3,* MEMBERS WITH ASSIGNED PLASTIC SECTION
     $ MODULI:*,16I4)
      READ 20,COP,CC,CE
      PRINT 50,COP,CC,CE
   50 FORMAT (//2X,*ASSIGNED COLUMN PLASTIC SECTION MODULUS:*//5X,
     $*DIVIDING PT.---*,F8.2//5X,*COEFFICIENT---*,2F7.2//5X,*EXPONENT---
     $*,2F8.3)
   52 READ 20,RCS,RCD,BFS
      PRINT 54,RCS,RCD,BFS
   54 FORMAT (//2X,*STATIC LOADING REDUCTION COEFFICIENT---*,F7.2//2X,
     $*DYNAMIC LOADING REDUCTION COEFFICIENT---*,F7.2//2X,*BUCKLING REDU
     $CTION COEFFICIENT---*,F7.2)
      READ 20,DA
      PRINT 56,(DA(I),I=1,NSTR)
   56 FORMAT (/2X,*FLOOR DISPLACEMENT ALLOWABLES (TOP DOWN)---*,8F8.3)
      READ 20,(DUCA(I),I=1,NCS2)
      PRINT 58,(DUCA(I),I=1,NCS2)
   58 FORMAT (/2X,*ALLOWABLE DUCTILITY (TOP DOWN)---*,16F6.1)
      CSMP=CSM+CTS
      CSCP=CSC+CTS
      COH=(1.+COH)*(1.+PFT)
      NCNST=4*NCS5+NCS2
      CALL DESIGN(B,NCNST)
      STOP
      END
```

```
      SUBROUTINE DESIGN(B,NCNST)
      COMMON /STDAT/NSTR,W(8),ELB,FL(8),Q,NCS2,NCS3,NCS4,NCS5,NCS6,NCSL,
     $NCO
      COMMON /DESINF/NCC,MC(38),MUC(39),NSC,MS(16),COP,CC(2),CE(2),X(40)
      DIMENSION B(1),KPC(38),DELB(40),G(40),DELC(40),BU(16),BL(16),KAC(6
     $),GF(40),H(16),BN(16),GN(40),C(178),KVC(10)
      EQUIVALENCE (MC,KPC),(BSUL,KSW),(BSLL,KE),(BSM,M),(CSUL,NV),
     $(CSLL,FD,K2),(CSM,EPS,T,K3),(BAC,N,DBN,KA),(BMAX,FN,K5),(BMIN,FOD,
     $K4),(CMAX,FNO),(CMIN,KAC,KVC),(KCL,KAC(2)),(SU,KAC(3)),(SL,KAC(4))
     $,(GM,KAC(5)),(KNL,KAC(6)),(BL,H),(DELB,GF,C),(G,C(41)),(GN,C(81)),
     $(DELC,C(121)),(BN,C(161))
      FO=F(B,DELB,0,1)
      NF=1
      READ 1100,BSUL,BSLL,BSM,CSUL,CSLL,CSM
      PRINT 1200,BSUL,BSLL,BSM,CSUL,CSLL,CSM
      READ 1100,BAC,EPSM,EPSD,BHL,CHL,RHO
      PRINT 1201,BAC,EPSM,EPSD,BHL,CHL
      PRINT 1202,RHO
      READ 1110,KCL,KNL,KL,NGL
      PRINT 1203,KCL,KNL,KL,NGL
      READ 1110,(KPC(J),J=1,16)
      DO 2 J=1,16
      IF(KPC(J).EQ.0) GO TO 6
    2 CONTINUE
      READ 1110,(KPC(J),J=17,32)
      DO 3 J=17,32
      IF(KPC(J).EQ.0) GO TO 6
    3 CONTINUE
      READ 1110,(KPC(J),J=33,38)
      DO 4 J=33,38
      IF(KPC(J).EQ.0) GO TO 6
    4 CONTINUE
    6 KPC(J)=200
      KN=J-1
      PRINT 1204,(KPC(K),K=1,KN)
      BMAX=3000.
      BMIN=150.
      CMAX=2000.
      CMIN=150.
      PRINT 1205,BMAX,BMIN,CMAX,CMIN
      S=0.0
      DO 7 J=1,NCO
      DELC(J)=0.0
      K=MUC(J)
      IF(MOD(K,2)) 70,71,70
   70 S=S+BHL*BHL
      GO TO 7
   71 S=S+CHL*CHL
    7 CONTINUE
      HLIM=SQRT(S)
      READ 1110,K
      IF(K.LE.1) GO TO 90
      CALL FDRIV(GF,0,0.)
      READ 1100,(BU(J),J=1,NCO)
      KC=0
      PRINT 1002,KC,(BU(J),J=1,NCO)
      M=0
      IF(K.EQ.2) GO TO 100
      KSW=2
      KE=1
      GO TO 101
   90 PRINT 1000
      KN=0
      GO TO 9
    8 FO=F(B,DELB,0,1)
      NF=NF+1
    9 PRINT 1001,FO
      CALL FDRIV(G,0,0.)
      DO 15 J=1,NCO
      BL(J)=0.0
      BU(J)=5000.
      K=MUC(J)
      IF(MOD(K,2)) 10,12,10
   10 SU=BSUL
      SL=BSLL
      SM=BSM
      GO TO 14
   12 SU=CSUL
      SL=CSLL
      SM=CSM
   14 DB=-SM*G(J)
      AD=ABS(DB)
      IF(AD.GT.SU) DB=SIGN(SU,DB)
      IF(AD.LT.SL) DB=SIGN(SL,DB)
   15 DELB(J)=DB
      KN=KN+1
      KC=0
   16 KM=0
      DO 30 J=1,NCO
      DELC(J)=0.0
      IF(BU(J)-BL(J).LE.BAC) GO TO 29
      IF(G(J)) 19,20,21
   19 BL(J)=B(J)
      GO TO 22
   20 BL(J)=B(J)-5.
      BU(J)=B(J)+5.
      GO TO 22
   21 BU(J)=B(J)
   22 IF(BL(J).NE.0.0.AND.BU(J).NE.5000.) GO TO 27
      K=MUC(J)
      IF(MOD(K,2)) 24,25,24
   24 SU=BMAX
      SL=BMIN
      GO TO 26
   25 SU=CMAX
      SL=CMIN
   26 IF(BL(J).GE.SU) GO TO 900
      IF(BU(J).LE.SL) GO TO 900
      B(J)=B(J)+DELB(J)
      IF(B(J).LT.SL) B(J)=SL
      IF(B(J).GT.SU) B(J)=SU
```

II

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      GO TO 30
   27 B(J)=(BU(J)+BL(J))/2.
      IF(BU(J)-BL(J).LE.BAC) KB=KB+1
      GO TO 30
   29 KB=KB+1
   30 CONTINUE
      KC=KC+1
      IF(KB.EQ.NCD) GO TO 35
      IF(KC.EQ.KCL) GO TO 901
      FO=F(B,DELC,0,1)
      NF=NF+1
      CALL FDRIV(G,0,0.)
      GO TO 18
   35 PRINT 1002,KC,(B(J),J=1,NCD)
      IF(KN.EQ.KNL) GO TO 50
      FO=F(BU,DELC,0,1)
      NF=NF+1
      CALL FDRIV(G,0,0.)
      DO 38 K=1,NCD
   38 DELB(K)=0.0
      DO 40 J=1,NCD
      IF(G(J).GT.0.) GO TO 40
      PRINT 1003
      GO TO 8
   40 CONTINUE
      FO=F(BL,DELB,0,1)
      NF=NF+1
      CALL FDRIV(G,0,0.)
      DO 45 J=1,NCD
      IF(G(J).LT.0.) GO TO 45
      PRINT 1003
      DO 42 K=1,NCD
   42 DELB(K)=0.0
      GO TO 9
   45 CONTINUE
      DO 47 K=1,NCD
   47 DELC(K)=0.0
   50 DO 52 K=1,NCD
   52 BU(K)=B(K)
      PRINT 1300,NF
      M=1
      NF=NF+1
  100 KSW=0
      KE=1
      PRINT 1010
  101 KN=0
      NV=0
      KC=0
  102 KN=KN+1
      KB=KPC(KN)
      IF(KB.GT.NCSL) GO TO 110
      FD=F(B,DELC,KB,M)
      M=0
      EPS=EPSM
      IF(KB.GT.NCS4+NCS4) EPS=EPSO
      IF(FD+EPS/10..GT.0.) GO TO 104
      IF(FD+EPS.LT.0.) GO TO 102
      KC=KC+1
      KAC(KC)=KB
      CALL FDRIV(G,KB,0.)
      CALL AM(G,KC,NCD)
      GO TO 102
  104 PRINT 1011,KB
      IF(NV.NE.0) GO TO 102
      NV=KB
      CALL FDRIV(GF,KB,0.)
      FO=FD
      AD=EPS/2.
      GO TO 102
  110 IF(KN.EQ.1.AND.M.EQ.1) FD=F(B,DELC,2*NCS4+1,1)
      M=-1
      GO TO 117
  115 KN=KN+1
      KB=KPC(KN)
  117 IF(KB.GT.NCSL+NCS6) GO TO 121
      FD=F(B,DELC,KB,M)
      M=0
      IF(FD+EPSM/10..GT.0.) GO TO 119
      IF(FD+EPSM.LT.0.) GO TO 115
      KC=KC+1
      KAC(KC)=KB
      CALL FDRIV(G,KB,0.)
      CALL AM(G,KC,NCD)
      GO TO 115
  119 PRINT 1011,KB
      IF(NV.NE.0) GO TO 115
      NV=KB
      CALL FDRIV(GF,KB,0.)
      FO=FD
      AD=EPSM/2.
      GO TO 115
  121 M=-1
      GO TO 125
  123 KN=KN+1
      KB=KPC(KN)
  125 IF(KB.GT.2*NCSL+NCS2) GO TO 150
      FD=F(B,DELC,KB,M)
      M=0
      IF(FD+EPSM/10..GT.0.) GO TO 127
      IF(FD+EPSM.LT.0.) GO TO 123
      KC=KC+1
      KAC(KC)=KB
      CALL FDRIV(G,KB,0.)
      CALL AN(G,KC,NCD)
      GO TO 123
  127 PRINT 1011,KB
      IF(NV.NE.0) GO TO 123
      NV=KB
      CALL FDRIV(GF,KB,0.)
      FO=FD
      AD=EPSM/2.
      GO TO 123
  150 IF(NV.EQ.0) GO TO 200
      S=0.0
      IF(KSW.EQ.3) KSW=4
      DO 152 J=1,NCD
      G(J)=0.0
      IF(GF(J).GE.S) GO TO 152
      N=J
      S=GF(J)
  152 CONTINUE
      G(N)=-1.
      CALL PROJ(KC,H,G,NCD)
      CALL EXTP(FO,AO,GF,H,NCD,DB)
      K=0
  154 DO 155 J=1,NCD
  155 BN(J)=B(J)+DB*H(J)
      M=1
      NF=NF+1
      IF(NV.LE.NCSL) GO TO 157
      FD=F(BN,DELC,2*NCS4+1,1)
      M=-1
  157 FD=F(BN,DELC,NV,M)
      IF(FD.LE.-AD/5..AND.FD.GE.-2.*AD) GO TO 165
      K=K+1
      IF(K.EQ.KL) GO TO 910
      CALL FDRIV(GN,NV,0.)
      CALL INTP(FD,GN,FO,GF,H,AD,DB,NCD)
      DO 160 J=1,NCD
      B(J)=BN(J)
  160 GF(J)=GN(J)
      FO=FD
      GO TO 154
  165 M=0
      DO 167 J=1,NCD
  167 B(J)=BN(J)
      PRINT 1012,NV,(BN(J),J=1,NCD)
      PRINT 1300,NF
      GO TO 101
  200 KAC(KC+1)=0
      IF(KSW.GE.3) GO TO 290
      IF(KC.EQ.0) GO TO 300
      IF(KSW.EQ.2) GO TO 300
      DO 202 J=1,NCD
  202 DELB(J)=B(J)-BU(J)
      CALL PROJ(KC,H,DELB,NCD)
      S=0.0
      DO 203 J=1,NCD
  203 S=S+H(J)**2
      S=SQRT(S)
      IF(S.LE.10.) GO TO 300
      M=1
      NF=NF+1
  204 KN=0
      K=1
  205 KN=KN+1
      KB=KPC(KN)
      IF(KB.GT.NCSL) GO TO 210
      IF(KB.NE.KAC(K)) GO TO 207
      K=K+1
      GO TO 205
  207 FD=F(B,H,KB,M)
      M=0
      EPS=EPSM
      IF(KB.GT.NCS4+NCS4) EPS=EPSO
      IF(FD+EPS/10..GT.0.) GO TO 250
      GO TO 205
  210 IF(KN.EQ.1.AND.M.EQ.1) FD=F(B,H,2*NCS4+1,1)
      EPS=EPSM
      M=-1
      GO TO 214
  212 KN=KN+1
      KB=KPC(KN)
  214 IF(KB.GT.NCSL+NCS6) GO TO 218
      IF(KB.NE.KAC(K)) GO TO 216
      K=K+1
      GO TO 212
  216 FD=F(B,H,KB,M)
      M=0
      IF(FD+EPSM/10..GT.0.) GO TO 250
      GO TO 212
  218 M=-1
      GO TO 222
  220 KN=KN+1
      KB=KPC(KN)
  222 IF(KB.GT.2*NCSL+NCS2) GO TO 230
      IF(KB.NE.KAC(K)) GO TO 224
      K=K+1
      GO TO 220
  224 FD=F(B,H,KB,M)
      M=0
      IF(FD+EPSM/10..GT.0.) GO TO 250
      GO TO 220
  230 IF(KSW.EQ.0) KSW=2
      DO 232 J=1,NCD
  232 B(J)=B(J)+H(J)
      IF(KSW.LE.2) GO TO 240
      PRINT 1014,(B(J),J=1,NCD)
      PRINT 1300,NF
      GO TO 101
  240 PRINT 1013,(B(J),J=1,NCD)
      PRINT 1300,NF
      IF(KSW.EQ.1) KSW=0
      GO TO 101
  250 CALL FDRIV(G,KB,0.)
      FO=FD
      CALL EXTP(FD,EPS/2.,G,H,NCD,DB)
      K=0
  254 DO 255 J=1,NCD
  255 H(J)=(1.+DB)*H(J)
      M=1
      NF=NF+1
      IF(KB.LE.NCSL) GO TO 257
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III

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      FD=F(B,H,2*NCS4+1,1)
      M=-1
257   FO=F(B,H,KB,M)
      IF(FD.LE.-EPS/10..AND.FD.GE.-EPS) GO TO 265
      K=K+1
      IF(K.EQ.KL) GO TO 930
      CALL FDRIV(GN,KB,0.)
      DB=DB/(1.+DB)
      CALL INTP(FD,GN,FO,G,H,EPS/2.,DB,NCD)
      DO 260 J=1,NCD
260   G(J)=GN(J)
      FD=FO
      GO TO 254
265   M=0
      IF(KSW.LE.2) GO TO 267
      IF(DBN.EQ.0.) RHO=RHO/2.
      DBN=1.
      KSW=4
      GO TO 204
267   KSW=1
      GO TO 204
290   IF(KSW.EQ.4) GO TO 302
      FO=FN
      DO 292 J=1,NCD
292   G(J)=GN(J)
      PRINT 1021,FO
      GO TO 304
300   PRINT 1020
      NG=0
302   FO=F(B,DELC,0,0)
      PRINT 1021,FO
      CALL FDRIV(G,0,0.)
304   CALL PROJ(KC,H,G,NCD)
      S=0.0
      DO 305 J=1,NCD
      DELC(J)=0.0
      DELB(J)=0.0
305   S=S+H(J)**2
      S=SQRT(S)
      IF(S.GT.HLIM) GO TO 309
      DO 308 J=1,NCD
      K=MUC(J)
      IF(MOD(K,2)) 306,307,306
306   IF(H(J).GT.BHL) GO TO 309
      GO TO 308
307   IF(H(J).GT.CHL) GO TO 309
308   CONTINUE
      IF(KF.EQ.2) GO TO 400
      KF=2
      EPSM=EPSM/10.
      EPSD=EPSD/10.
      FN=FO
      DO 340 J=1,NCD
340   GN(J)=G(J)
      KSW=3
      RHO=10.
      M=0
      PRINT 1022
      GO TO 101
309   DB=RHO/S
      NG=NG+1
      IF(NG.EQ.NGL) GO TO 940
      DO 310 J=1,NCD
310   BN(J)=B(J)+DB*H(J)
      FD=F(BN,DELB,0,1)
      NF=NF+1
      CALL FDRIV(GN,0,0.)
      S=0.0
      DO 315 J=1,NCD
315   S=S+G(J)*H(J)
      FOD=S
      S=0.0
      DO 320 J=1,NCD
320   S=S+GN(J)*H(J)
      FND=S
      IF(FOD*FND.GE.0.0) GO TO 322
      S=FOD+FND+3.*(FO-FD)/DB
      T=SQRT(S*S-FOD*FND)
      DBN=-DB*(FND+T-S)/(FND-FOD+2.*T)
      M=1
      KSW=4
      NF=NF+1
      IF(DBN.LE.0.) GO TO 325
      NF=NF-1
322   DBN=0.0
      M=0
      FN=FD
      KSW=3
325   RHO=2.*(DB+DBN)*RHO/DB
      DB=DB+DBN
      DO 327 J=1,NCD
327   H(J)=DB*H(J)
      GO TO 204
400   PRINT 1030
      NV=0
      KN=0
      K=1
405   KN=KN+1
      IF(KN.GT.NCSL) GO TO 410
      IF(KN.NE.KPC(K)) GO TO 407
      K=K+1
407   FD=F(B,DELB,KN,0)
      C(KN)=FD
      IF(FD.LE.0.) GO TO 405
      NV=NV+1
      KVC(NV)=KN
      PRINT 1011,KN
      GO TO 405
410   M=-1
      GO TO 414
412   KN=KN+1
414   IF(KN.GT.NCSL+NCS6) GO TO 420
      IF(KN.NE.KPC(K)) GO TO 416
      K=K+1
416   FD=F(B,DELB,KN,M)
      C(KN)=FD
      M=0
      IF(FD.LE.0) GO TO 412
      NV=NV+1
      KVC(NV)=KN
      PRINT 1011,KN
      GO TO 412
420   M=-1
      GO TO 424
422   KN=KN+1
424   IF(KN.GT.2*NCSL+NCS2) GO TO 430
      IF(KN.NE.KPC(K)) GO TO 426
      K=K+1
426   FD=F(B,DELB,KN,M)
      C(KN)=FD
      M=0
      IF(FD.LE.0) GO TO 422
      NV=NV+1
      KVC(NV)=KN
      PRINT 1011,KN
      GO TO 422
430   IF(NV.EQ.0) GO TO 500
435   KPC(K+NV)=KPC(K)
      K=K-1
440   IF(KPC(K).GT.KVC(NV)) GO TO 435
      KPC(K+NV)=KVC(NV)
      NV=NV-1
      IF(NV.NE.0) GO TO 440
      EPSM=10.*EPSM
      EPSD=10.*EPSD
      RHO=20.
      PRINT 1035
      DO 450 J=1,NCD
450   DELC(J)=0.0
      GO TO 100
500   PRINT 1036
      PRINT 1090,(X(J),J=1,NCS2)
      K=NCS2+1
      PRINT 1091,(X(J),J=K,NCS3)
      K=NCS3+1
      PRINT 1092,(X(J),J=K,NCS5)
      PRINT 1037,(KAC(J),J=1,KC)
      PRINT 1038
      KN=1
      NV=NCSL
      K=0
505   K=K+1
      IF(K.GT.NV) GO TO 515
      K2=K+4
      IF(K2.GT.NV) GO TO 550
      K3=K+8
      IF(K3.GT.NV) GO TO 555
      K4=K+12
      IF(K4.GT.NV) GO TO 560
      K5=K+16
      IF(K5.GT.NV)GO TO 565
      K6=K+20
      IF(K6.GT.NV) GO TO 570
      GO TO 575
510   IF(MOD(K,4).NE.0) GO TO 505
      PRINT 1050
      K=K+20
      GO TO 505
515   PRINT 1039
      KN=2
      K=NCSL
      NV=NCSL+NCS6
520   K=K+1
      IF(K.GT.NV) GO TO 530
      K2=K+4
      IF(K2.GT.NV) GO TO 550
      K3=K+8
      IF(K3.GT.NV) GO TO 555
      K4=K+12
      IF(K4.GT.NV) GO TO 560
      K5=K+16
      IF(K5.GT.NV)GO TO 565
      K6=K+20
      IF(K6.GT.NV) GO TO 570
      GO TO 575
525   KB=K-NCSL
      IF(MOD(KB,4).NE.0) GO TO 520
      PRINT 1050
      K=K+20
      GO TO 520
530   PRINT 1040
      KN=3
      K=NV
      NV=NV+NCS6
535   K=K+1
      IF(K.GT.NV) GO TO 600
      K2=K+4
      IF(K2.GT.NV) GO TO 550
      K3=K+8
      IF(K3.GT.NV) GO TO 555
      K4=K+12
      IF(K4.GT.NV) GO TO 560
      K5=K+16
      IF(K5.GT.NV)GO TO 565
      K6=K+20
      IF(K6.GT.NV) GO TO 570
      GO TO 575
540   KB=K-NCSL-NCS6
      IF(MOD(KB,4).NE.0) GO TO 535
      PRINT 1050
      K=K+20
      GO TO 535
550   PRINT 1041,K,C(K)
      GO TO (510,525,540),KN
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IV

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555 PRINT 1042,K,C(K),K2,C(K2)
    GO TO (510,525,540),KN
560 PRINT 1043,K,C(K),K2,C(K2),K3,C(K3)
    GO TO (510,525,540),KN
565 PRINT 1044,K,C(K),K2,C(K2),K3,C(K3),K4,C(K4)
    GO TO (510,525,540),KN
570 PRINT 1045,K,C(K),K2,C(K2),K3,C(K3),K4,C(K4),K5,C(K5)
    GO TO (510,525,540),KN
575 PRINT 1046,K,C(K),K2,C(K2),K3,C(K3),K4,C(X4),K5,C(K5),K6,C(K6)
    GO TO (510,525,540),KN
600 PRINT 1047
    RETURN
900 PRINT 1060,(B(J),J=1,NCD)
    RETURN
901 PRINT 1061,(B(J),J=1,NCD)
    RETURN
910 PRINT 1070,(BN(J),J=1,NCD)
    RETURN
930 DO 931 J=1,NCD
931 B(J)=B(J)+H(J)
    PRINT 1071,(B(J),J=1,NCD)
    RETURN
940 PRINT 1080
    RETURN
1000 FORMAT(///2X,89H********************************************** PHASE 1 *
    $****************************************/)
1001 FORMAT(/2X,*INITIAL DESIGN COST---*,F10.2)
1002 FORMAT(/2X,*AFTER*,I3,* ITERATIONS THE STATIONARY POINT IS ESTIMAT
    $ED TO BE AT:*/(2X,12F10.2))
1003 FORMAT(//2X,27H***** RESTART PHASE 1 *****)
1010 FORMAT(///2X,89H********************************************** PHASE 2 *
    $****************************************/)
1011 FORMAT(/2X,*CONSTRAINT NUMBER*,I4,* IS VIOLATED*)
1012 FORMAT(/2X,*SATISFACTION OF PREVIOUSLY VIOLATED CONSTRAINT*,I4,* Y
    $IELDS THE DESIGN VECTOR:*/(2X,12F10.2))
1013 FORMAT(/2X,*AN ADJUSTMENT STEP FROM THE PRECEDING DESIGN VECTOR YI
    $ELDS:*/(2X,12F10.2))
1014 FORMAT(//2X,*GRADIENT PROJECTION ITERATION YIELDS THE DESIGN VECTO
    $R:*/(2X,12F10.2))
1020 FORMAT(///2X,89H********************************************** PHASE 3 *
    $****************************************/)
1021 FORMAT(/2X,*PRESENT DESIGN COST---*,F10.2)
1022 FORMAT(/2X,*PREVIOUS DESIGN VECTOR IS OPTIMAL IN APPROXIMATE FEASI
    $BLE DOMAIN - BETTER APPROXIMATION BEING INITIATED*)
1030 FORMAT(///2X,89H********************************************** PHASE 4 *
    $****************************************/)
1035 FORMAT(/2X,*THE VIOLATED CONSTRAINTS NOTED ABOVE HAVE BEEN ADDED T
    $O THE PRIMARY CONSTRAINT LIST AND PHASE 2 IS BEING RESTARTED*)
1036 FORMAT(/2X,88H************************** THE PRECEDING DESIGN VECTOR
    $ IS OPTIMAL **************************)
1037 FORMAT(/2X,*ACTIVE CONSTRAINTS AT THE OPTIMAL ARE:*,10I5)
1038 FORMAT(//2X,*STATIC LOAD CONSTRAINT VALUES:*/)
1039 FORMAT(//2X,*MODERATE EARTHQUAKE CONSTRAINT VALUES:*/)
1040 FORMAT(//2X,*STRONG EARTHQUAKE CONSTRAINT VALUES:*/)
1041 FORMAT(4X,*G(*,I3,*)=*,F10.2)
1042 FORMAT(2(4X,*G(*,I3,*)=*,F10.2))
1043 FORMAT(3(4X,*G(*,I3,*)=*,F10.2))
1044 FORMAT(4(4X,*G(*,I3,*)=*,F10.2))
1045 FORMAT(5(4X,*G(*,I3,*)=*,F10.2))
1046 FORMAT(6(4X,*G(*,I3,*)=*,F10.2))
1047 FORMAT(//2X,130H****************************************** CONGRATULATION
    $S, YOU*VE MADE IT, NOW PAY THE COMPUTER BILL **********************
    $***************)
1050 FORMAT(/)
1060 FORMAT(//2X,*MEMBER SIZE RESTRICTIONS HAVE BEEN EXCEEDED -- THE PR
    $ESENT DESIGN VECTOR IS:*/(2X,12F10.2))
1061 FORMAT(//2X,*COORDINATE SEARCH ITERATION LIMIT EXCEEDED -- THE PRE
    $SENT DESIGN VECTOR IS:*/(2X,12F10.2))
1070 FORMAT(//2X,*VIOLATED CONSTRAINT SEARCH ITERATION LIMIT EXCEEDED -
    $- THE PRESENT DESIGN VECTOR IS:*/(2X,12F10.2))
1071 FORMAT(//2X,*CONSTRAINT SEARCH ITERATION LIMIT EXCEEDED -- THE PRE
    $SENT DESIGN VECTOR IS:*/(2X,12F10.2))
1080 FORMAT(//2X,*GRADIENT PROJECTION ITERATION LIMIT EXCEEDED*)
1090 FORMAT(//2X,*THE COMPLETE DESIGN VECTOR AT THE OPTIMAL POINT IS:*/
    $/2X,*MEMBER MOMENTS OF INERTIA (TOP DOWN)---*,8F11.2/40X,8F11.2)
1091 FORMAT(2X,*CONNECTION STIFFNESSES (TOP DOWN)---*,1P8E11.2)
1092 FORMAT(/2X,*MEMBER PLASTIC SECTION MODULI (TOP DOWN)---*,8F9.2/45X
    $,8F9.2/)
1100 FORMAT(8E10.0)
1110 FORMAT(16I5)
1200 FORMAT(//2X,*UNCONSTRAINED MINIMIZATION PARAMETERS:*//5X,*BEAM STE
    $P SIZE UPPER LIMIT---*,F8.1//5X,*BEAM STEP SIZE LOWER LIMIT---*,F8
    $.1//5X,*BEAM STEP MULTIPLIER---*,1PE11.2//5X,*COLUMN STEP SIZE UPP
    $ER LIMIT---*,0PF8.1//5X,*COLUMN STEP SIZE LOWER LIMIT---*,F8.1//5X
    $,*COLUMN STEP MULTIPLIER---*,1PE11.2)
1201 FORMAT(//2X,*ALGORITHM ACCURACY BOUNDS:*//5X,*UNCONSTRAINED MINIMI
    $ZATION DESIGN VARIABLE TOLERANCE---*,F7.1//5X,*MOMENT CONSTRAINTS
    $TOLERANCE---*,F7.1//5X,*DISPLACEMENT CONSTRAINTS TOLERANCE---*,
    $F7.3//5X,*ALGORITHM TERMINATION VALUE FOR BEAMS---*,1PE11.2//5X,
    $*ALGORITHM TERMINATION VALUE FOR COLUMNS---*,1PE11.2)
1202 FORMAT(//2X,*INITIAL STEP SIZE FOR GRADIENT PROJECTION---*,F8.1)
1203 FORMAT(//2X,*ALGORITHM ITERATION LIMITS:*//5X,*UNCONSTRAINED MINIM
    $UM SEARCH---*,I5//5X,*PHASE ONE RESTARTS---*,I5//5X,*CONSTRAINT FU
    $NCTION SEARCH---*,I5//5X,*CONSTRAINED MINIMUM SEARCH---*,I5)
1204 FORMAT(//2X,*PRIMARY CONSTRAINTS:*,20I5/22X,20I5)
1205 FORMAT(//2X,*MEMBER SIZE RESTRICTIONS TO INSURE MODELING ACCURACY
    $ARE:*//5X,*MAXIMUM BEAM SIZE---*,F8.1//5X,*MINIMUM BEAM SIZE--*,F8
    $.1//5X,*MAXIMUM COLUMN SIZE---*,F8.1//5X,*MINIMUM COLUMN SIZE---*,
    $F8.1)
1300 FORMAT(/2X,*NUMBER OF COMPLETE ANALYSES =*,I4)
    END
```

```
    SUBROUTINE AM(G,KC,NCD)
    DIMENSION G(1)
    COMMON /AMAT/A(5,16)
    DO 10 J=1,NCD
 10 A(KC,J)=G(J)
    RETURN
    END
```

```
    SUBROUTINE PROJ(KC,G,GF,NCD)
    COMMON /AMAT/A(5,16)
    DIMENSION G(1),GF(1),B(5),AA(5,5)
 20 IF(KC.EQ.0) GO TO 97
    DO 30 I=1,KC
    DO 30 J=1,KC
    S=0.0
    DO 25 K=1,NCD
 25 S=S+A(I,K)*A(J,K)
 30 AA(I,J)=S
    DO 60 N=1,KC
    D=AA(N,N)
    DO 35 J=1,KC
 35 AA(N,J)=-AA(N,J)/D
    DO 55 I=1,KC
    IF(N-I) 40,55,40
 40 DO 50 J=1,KC
    IF(N-J) 45,50,45
 45 AA(I,J)=AA(I,J)+AA(I,N)*AA(N,J)
 50 CONTINUE
 55 AA(I,N)=AA(I,N)/D
    AA(N,N)=1./D
 60 CONTINUE
    DO 75 I=1,KC
    T=0.0
    DO 70 J=1,NCD
    S=0.0
    DO 65 K=1,KC
 65 S=S+AA(I,K)*A(K,J)
 70 T=T+S*GF(J)
 75 B(I)=-T
    S=0.0
    DO 80 J=1,KC
    IF(B(J).GE.S) GO TO 80
    S=B(J)
    N=J
 80 CONTINUE
    IF(S.EQ.0.) GO TO 90
    KC=KC-1
    IF(N.EQ.KC+1) GO TO 20
    DO 85 J=N,KC
    DO 85 K=1,NCD
 85 A(J,K)=A(J+1,K)
    GO TO 20
 90 DO 95 J=1,NCD
    S=0.0
    DO 93 K=1,KC
 93 S=S+A(K,J)*B(K)
 95 G(J)=-S-GF(J)
    GO TO 100
 97 DO 98 J=1,NCD
 98 G(J)=-GF(J)
100 RETURN
    END
```

```
    SUBROUTINE EXTP(F,AD,G,H,NCD,DB)
    DIMENSION G(1),H(1)
    S=0.0
    DO 10 J=1,NCD
 10 S=S-G(J)*H(J)
    DB=(F+AD)/S
    RETURN
    END
```

```
    SUBROUTINE INTP(FN,GN,FO,GO,H,AD,DB,NCD)
    DIMENSION GN(1),GO(1),H(1)
    FNE=FN+AD
    S=0.0
    DO 5 J=1,NCD
  5 S=S+GN(J)*H(J)
    FND=S
    S=0.0
    DO 10 J=1,NCD
 10 S=S+GO(J)*H(J)
    FOD=S
    FP=-6.*(FN-FO)/(DB*DB)+2.*(2.*FND+FOD)/DB
    DB=-FNE/FND-FNE*FNE*FP/(2.*FND*FND*FND)
    RETURN
    END
```

```
    FUNCTION F(B,DELB,I,M)
    COMMON /STDAT/NSTR,W(8),ELB,EL(8),Q,NCS2,NCS3,NCS4,NCS5,NCS6,NCSL,
   $NCD
    COMMON /RES/SM(48),OMT(48),DELT(8)
    COMMON /COST/CSMP,CSCP,CSW,CPT,COM,CI,FP(8),NAG,AG(10),DL
    COMMON /DESINF/NCC,MC(38),MUC(39),NSC,MS(16),COP,CC(2),CE(2),X(40)
    COMMON /CNSTINF/RCS,DA(8),BFS,RCD,DEQ(2),DUCA(16)
    DIMENSION B(1),DELB(1)
    DATA KSWT/1/
    IF(KSWT) 2,15,2
  2 KSWT=0
    NCS4=4*NSTR
    NCS6=6*NSTR
    NCO=NCS5-NCC
```

V

```
      NCSL=10*NSTR
      DO 4 J=1,NCS5
    4 X(J)=B(J)
      IF(NCC.EQ.0) GO TO 18
      K=0
      DO 10 J=1,NCD
    6 L=J+K
      IF(L.EQ.MC(K+1)) GO TO 8
      B(J)=X(L)
      MUC(J)=L
      GO TO 10
    8 K=K+1
      GO TO 6
   10 CONTINUE
      GO TO 18
   15 IF(M.LE.0) GO TO 19
      IF(NCC.EQ.0) GO TO 16
  100 DO 102 J=1,NCD
      K=MUC(J)
  102 X(K)=B(J)+DELB(J)
      IF(NSC.EQ.0) GO TO 18
      DO 108 J=1,NSC
      K=MS(J)
      L=K+NCS3
      IF(MOD(K,2)) 104,106,104
  104 X(L)=.85*X(K)**.713
      GO TO 108
  106 KK=1
      IF(X(K).GE.COP) KK=2
      X(L)=CC(KK)*X(K)**CE(KK)
  108 CONTINUE
      GO TO 18
   16 DO 17 J=1,NCD
   17 X(J)=B(J)+DELB(J)
   18 CALL ANAL(X,0.0,0)
   19 IF(I.GT.0) GO TO 43
      S1=0.0
      S2=0.0
      S3=0.0
      S4=0.0
      S5=0.0
      DO 27 J=1,NCS2
      EI=X(J)
      EI2=EI*EI
      Z=X(J+NCS3)
      Z2=Z*Z
      A=1.1*Z2/EI
      IF(MOD(J,2)) 23,21,23
   21 ELL=EL(J/2)
      GO TO 25
   23 ELL=ELB
      S3=S3+Z2*(10800./X((J+1)/2+NCS2)+.6*Z2/EI2)
      S4=S4+Z2*(.73+.0026*Z2/EI+.064*Z/SQRT(EI))/EI
   25 S1=S1+A*ELL
      S2=S2+ELL*A**.79
   27 S5=S5+EI*ELL*(4.7/(2.09-.81*Z2*Z2/(EI2*EI))+2.26)/Z
      S=.002833*(CSMP*S1+.916*S2+.72*CSCP*S3+CSW*S4)+2.*CPT*S5
      IF(CI.GT.0.0) GO TO 28
      CI=-S*CI
      PRINT 200,CI
  200 FORMAT (//2X,*INITIAL COST TIMES ( ONE PLUS DOWN TIME MULTIPLIER )
     $---*,F10.2)
   28 S=S*COH
C       DAMAGE ESTIMATE
      NAG1=NAG-1
      DT=0.0
      DO 31 J=2,NAG1
      KSTR=0
      IF(AG(J).GT..25) KSTR=1
      CALL ANAL(X,AG(J),KSTR)
      DN=0.0
      DO 29 K=1,NSTR
   29 DN=DN+FP(K)*DELT(K)
      EN=.714
C         DS=0 ASSUMED
      EN=3.44*EXP(-15.25*AG(J))*DL*EN
      EN=EN*(AG(J+1)-AG(J-1))
      DT=DN*EN+DT
      CALL FDRIV(DELB,-1,EN)
   31 CONTINUE
      DT=DT*CI/2.
      F=S+DT
      RETURN
   43 IF(I.GT.NCSL) GO TO 72
      IF(I.GT.NCS4) GO TO 50
      K=I-1
      KM=MOD(K,4)
      MN=(K-KM)/2+1
      Z=X(MN+NCS3)
      N=KM+1
      GO TO (45,46,47,48),N
   45 F=SM(MN)-W((MN+1)/2)*ELB/12.-RCS*36.*Z
      RETURN
   46 F=-SM(MN)+W((MN+1)/2)*ELB/12.-RCS*36.*Z
      RETURN
   47 F=SM(MN)+W((MN+1)/2)*ELB/24.-RCS*36.*Z
      RETURN
   48 F=-SM(MN)-W((MN+1)/2)*ELB/24.-RCS*36.*Z
      RETURN
   50 IF(I.GT.2*NCS4) GO TO 59
      K=I-NCS4-1
      KM=MOD(K,4)
      MN=(K-KM)/2+2
      Z=X(MN+NCS3)
      N=KM+1
      MN2=MN/2
      P=0.0
      DO 52 J=1,MN2
   52 P=P+W(J)/2.
      NN=(MN2-1)*4+NCS2
      PR=P*X(MN)/(1.1*Z*Z*36.)
      PM=36.*Z
      IF(PR.GE..15) PM=1.18*PM*(1.-PR)
      GO TO (54,55,56,57),N
   54 F=SM(NN+1)-RCS*PM
      RETURN
   55 F=-SM(NN+1)-RCS*PM
      RETURN
   56 F=SM(NN+2)-RCS*PM
      RETURN
   57 F=-SM(NN+2)-RCS*PM
      RETURN
   59 IF(I.GT.NCS6+NCS3) GO TO 61
      N=I-NCS4-NCS4
      MN=2*N-1
      F=ELB*ELB*(W(N)*ELB/384.+SM(MN)/8.)/(Q*X(MN))-DA(N)
      RETURN
   61 N=I-NCS4-NCS5
      MN=2*N
      CR=0.0
      IF(N.EQ.1) GO TO 63
      CR=X(MN-2)/EL(N-1)
   63 CR=CR+X(MN)/EL(N)
      ELA=ELB+3.*Q*X(MN-1)/X(NCS2+N)
      BR=X(MN-1)/ELB*3./(4.*ELA/ELB-ELB/ELA)
      GA=CR/BR
      GB=1.
      IF(N.EQ.NSTR) GO TO 65
      CR=X(MN+2)/EL(N+1)+X(MN)/EL(N)
      ELA=ELB+3.*Q*X(MN+1)/X(NCS2+N+1)
      BR=X(MN+1)/ELB*3./(4.*ELA/ELB-ELB/ELA)
      GB=CR/BR
   65 SLR=3.1416/ROOT(GA,GB)
      Z=X(MN+NCS3)
      A=1.1*Z*Z/X(MN)
      RG=SQRT(X(MN)/A)
      SLR=SLR*EL(N)/RG
      IF(SLR.GE.128.25) GO TO 67
      PCR=36.*A*(1.-9.*SLR*SLR/Q/9.8696)
      GO TO 69
   67 PCR=9.8696*Q*A/(SLR*SLR)
   69 P=0.0
      DO 70 J=1,N
   70 P=P+W(J)/2.
      F=P-BFS*PCR
      RETURN
   72 K=I-NCSL
      L=1
      KSTR=0
      DMM=RCD
      IF(K.LE.NCS6) GO TO 74
      K=K-NCS6
      L=2
      KSTR=1
   74 IF(M.LT.0) CALL ANAL(X,DEQ(L),KSTR)
      IF(K.GT.NCS2) GO TO 78
      KM=MOD(K,2)
      N=(K+KM)/2
      MN=2*N-1
      Z=X(MN+NCS3)
      IF(L.EQ.2) DMM=DUCA(MN)
      WL2=W(N)*ELD/2.
      IF(KM.EQ.0) GO TO 76
      IF(WL2/2..GT.DMT(MN)) GO TO 75
      F=DMT(MN)+SM(MN)-WL2/6.-DMM*36.*Z
      RETURN
   75 F=SM(MN)+DMT(MN)*DMT(MN)/WL2+WL2/12.-DMM*36.*Z
      RETURN
   76 F=DMT(MN)-SM(MN)+WL2/6.-DMM*36.*Z
      RETURN
   78 KK=K-NCS2-1
      KM=MOD(KK,4)
      MN=(KK-KM)/2+2
      Z=X(MN+NCS3)
      IF(L.EQ.2) DMM=DUCA(MN)
      N=KM+1
      MN2=MN/2
      P=0.0
      DO 80 J=1,MN2
   80 P=P+W(J)/2.
      PR=P*X(MN)/(1.1*Z*Z*36.)
      PM=36.*Z
      IF(PR.GE..15) PM=1.18*PM*(1.-PR)
      GO TO (82,83,84,85),N
   82 F=DMT(K)+SM(K)-DMM*PM
      RETURN
   83 F=DMT(K-1)-SM(K-1)-DMM*PM
      RETURN
   84 F=DMT(K-1)+SM(K-1)-DMM*PM
      RETURN
   85 F=DMT(K-2)-SM(K-2)-DMM*PM
      RETURN
      END


      FUNCTION ROOT(GA,GB)
      J=1
      R=2.42
      GG=GA*GB
      GPG=GA+GB
    5 TR=TAN(R)
      TR2=TR*TR
      SR2=1.+TR2
      FR=(GG*R*R-36.)/6./GPG-R/TR
      DFR=GG*R/3./GPG+(R*SR2-TR)/TR2
      RN=R-FR/DFR
      IF(R-RN.LE..001) GO TO 10
      IF(J.GE.50) GO TO 15
      J=J+1
      R=RN
      GO TO 5
```

VI

```
   10 ROOT=RN
      RETURN
   15 PRINT 20
   20 FORMAT (/5X,*ROOT ITER. LIMIT EXCEEDED*)
      STOP
      END



      SUBROUTINE ANAL(B,GDAC,KSTR)
      COMMON /STDAT/NSTR,W,ELB,EL,Q,NBCM,NBCC,NDRF,NCDO,NCS6,NCSL,NCD
      COMMON /DEIG/E,V,DE,DV
      COMMON /RES/SM,DMT,DELT
      COMMON /ENDRV/Y,EM,DS,SR,RD,RXT,DM,DEL
      DIMENSION B(1),W(8),EL(8),EM(8),SR(32,32),SRX(32,8),G(8,8),R(33),D
     $S(36),SM(48),RXT(32,8),E(8),V(8,8),Y(8),X(8,8),DEL(8,8),A(8),C(8),
     $V1(8),V2(9),V3(8),V4(8),RD(32,8),DM(48,8),DMT(48),DELT(8),DE(8,24)
     $,DV(8,24,8)
      LOGICAL V4
      DATA KSWT/1/
      IF(KSWT) 5,15,5
C     FORM MASS MATRIX
    5 KSWT=0
      DO 10 I=1,NSTR
   10 EM(I)=W(I)/386.1
      Q=30000.
   15 IF(GDAC.GT.0.0) GO TO 92
C     FORM TOTAL STIFFNESS MATRIX
   20 DO 25 I=1,NDRF
      DO 24 J=1,NDRF
   24 SR(I,J)=0.0
      DO 25 J=1,NSTR
   25 SRX(I,J)=0.0
      DO 27 K=1,NSTR
      DO 27 J=1,NSTR
   27 G(K,J)=0.0
      IS=NBCM-1
      DO 30 I=1,IS,2
      B1=2.*Q*B(I)/ELB
      J=2*I
      J1=J+1
      SR(J,J)=SR(J,J)+2.*B1
      SR(J1,J1)=SR(J1,J1)+2.*B1
      SR(J,J1)=SR(J,J1)+B1
   30 SR(J1,J)=SR(J1,J)+B1
      IS=IS-1
      IF(NSTR.EQ.1) GO TO 40
      DO 35 I=2,IS,2
      K1=I/2
      K6=K1
      K7=K6+1
      K2=2*I-3
      K3=K2+3
      K4=K2+4
      K5=K2+7
      B1=2.*Q*B(I)/EL(K1)
      B2=3.*B1/EL(K1)
      SR(K2,K2)=SR(K2,K2)+2.*B1
      SR(K3,K3)=SR(K3,K3)+2.*B1
      SR(K4,K4)=SR(K4,K4)+2.*B1
      SR(K5,K5)=SR(K5,K5)+2.*B1
      SR(K2,K4)=SR(K2,K4)+B1
      SR(K4,K2)=SR(K4,K2)+B1
      SR(K3,K5)=SR(K3,K5)+B1
      SR(K5,K3)=SR(K5,K3)+B1
      SRX(K2,K6)=SRX(K2,K6)-B2
      SRX(K2,K7)=SRX(K2,K7)+B2
      SRX(K3,K6)=SRX(K3,K6)-B2
      SRX(K3,K7)=SRX(K3,K7)+B2
      SRX(K4,K6)=SRX(K4,K6)-B2
      SRX(K4,K7)=SRX(K4,K7)+B2
      SRX(K5,K6)=SRX(K5,K6)-B2
      SRX(K5,K7)=SRX(K5,K7)+B2
      B3=4.*B2/EL(K1)
      G(K6,K6)=G(K6,K6)+B3
      G(K7,K7)=G(K7,K7)+B3
      G(K6,K7)=G(K6,K7)-B3
   35 G(K7,K6)=G(K7,K6)-B3
   40 B1=2.*Q*B(NBCM)/EL(NSTR)
      B2=3.*B1/EL(NSTR)
      B3=4.*B2/EL(NSTR)
      K1=NDRF-3
      SR(K1,K1)=SR(K1,K1)+2.*B1
      SR(NDRF,NDRF)=SR(NDRF,NDRF)+2.*B1
      SRX(K1,NSTR)=SRX(K1,NSTR)-B2
      SRX(NDRF,NSTR)=SRX(NDRF,NSTR)-B2
      G(NSTR,NSTR)=G(NSTR,NSTR)+B3
      DO 45 I=1,NSTR
      J=NBCM+I
      K1=4*I-3
      K2=K1+1
      K3=K1+2
      K4=K1+3
      SR(K1,K1)=SR(K1,K1)+B(J)
      SR(K2,K2)=SR(K2,K2)+B(J)
      SR(K3,K3)=SR(K3,K3)+B(J)
      SR(K4,K4)=SR(K4,K4)+B(J)
      SR(K1,K2)=SR(K1,K2)-B(J)
      SR(K2,K1)=SR(K2,K1)-B(J)
      SR(K3,K4)=SR(K3,K4)-B(J)
   45 SR(K4,K3)=SR(K4,K3)-B(J)
C     FORM KR INVERSE
      DO 65 N=1,NDRF
      D=SR(N,N)
      DO 50 J=1,NDRF
   50 SR(N,J)=-SR(N,J)/D
      DO 63 I=1,NDRF
      IF(N-I) 55,63,55
   55 DO 60 J=1,NDRF
      IF(N-J) 58,60,58
   58 SR(I,J)=SR(I,J)+SR(I,N)*SR(N,J)
   60 CONTINUE
   63 SR(I,N)=SR(I,N)/D
      SR(N,N)=1.0/D
   65 CONTINUE
C     COMPLETE STATIC ANALYSIS
      DO 70 I=1,NDRF,4
      J=(I+3)/4
      R(I)=0.0
      R(I+1)=W(J)*ELB/12.
      R(I+2)=-R(I+1)
   70 R(I+3)=0.0
      CALL MATMUL(DS,SR,R,NDRF,NDRF,1,36,32,33)
      DO 72 I=1,4
   72 DS(NDRF+I)=0.0
      DO 75 I=1,NBCM,2
      B1=2.*Q*B(I)/ELB
      B2=2.*Q*B(I+1)/EL((I+1)/2)
      M=NBCM+2*I-1
      J=2*I
      J1=J+1
      K1=J-1
      K2=K1+4
      K3=K2-1
      K4=K3+4
      SM(I)=(2.*DS(J)+DS(J1))*B1
      SM(I+1)=(DS(J)+2.*DS(J1))*B1
      SM(M)=(2.*DS(K1)+DS(K2))*B2
      SM(M+1)=(DS(K1)+2.*DS(K2))*B2
      SM(M+2)=(2.*DS(K3)+DS(K4))*B2
   75 SM(M+3)=(DS(K3)+2.*DS(K4))*B2
C     FORM LATERAL STIFFNESS MATRIX
      CALL MATMUL(RXT,SR,SRX,NDRF,NDRF,NSTR,32,32,32)
      DO 80 I=1,NSTR
      DO 80 J=1,NSTR
      DO 80 K=1,NDRF
   80 G(I,J)=G(I,J)-SRX(K,I)*RXT(K,J)
C     SOLVE EIGEN-PROBLEM
      DO 85 I=1,NSTR
      DO 85 J=1,NSTR
   85 G(I,J)=G(I,J)/SQRT(EM(I)*EM(J))
      CALL HQRW(NSTR,8,-NSTR,G,E,V,A,C,V1,V2,V3,V4)
      DO 90 I=1,NSTR
      DO 90 J=1,NSTR
   90 V(J,I)=V(J,I)/SQRT(EM(J))
      CALL NORMV(V,EM,NSTR)
      GO TO 122
C     COMPLETE DYNAMIC ANALYSIS
   92 DO 100 I=1,NSTR
      FQ=SQRT(E(I))
      IF(KSTR.EQ.1) GO TO 94
      SD=36.*GDAC*1.4
      SV=48.*GDAC*1.9
      SA=386.*GDAC*2.6
      IF(FQ.GE.1.81) SD=SV/FQ
      IF(FQ.GT.11.) SD=SA/FQ**2.
      GO TO 96
   94 SD=36.*GDAC*1.1
      SV=48.*GDAC*1.3
      SA=386.*GDAC*1.5*1.51
      IF(FQ.GE.1.58) SD=SV/FQ
      IF(FQ.GT.14.01) SD=SA/FQ**2.
   96 S=0.0
      DO 98 J=1,NSTR
   98 S=S+V(J,I)*EM(J)
  100 Y(I)=S*SD
      DO 110 I=1,NSTR
      DO 102 J=1,NSTR
  102 X(J,I)=V(J,I)*Y(I)
      IS=NSTR-1
      IF(NSTR.EQ.1) GO TO 105
      DO 104 J=1,IS
  104 DEL(J,I)=X(J,I)-X(J+1,I)
  105 DEL(NSTR,I)=X(NSTR,I)
      DO 106 J=1,NSTR
  106 V1(J)=X(J,I)
      CALL MATMUL(R,RXT,V1,NDRF,NSTR,1,33,32,8)
      DO 108 J=1,NDRF
  108 RD(J,I)=R(J)
      R(NDRF+1)=0.0
      DO 110 J=1,NBCM,2
      B1=2.*Q*B(J)/ELB
      K=(J+1)/2
      B2=2.*Q*B(J+1)/EL(K)
      M=NBCM+2*J-1
      J1=2*J
      J2=J1+1
      K1=J1-1
      K2=K1+4
      VD=-R(K1)-DEL(K,I)/EL(K)
      VT=-R(K2)-DEL(K,I)/EL(K)
      DM(J,I)=-(2.*R(J1)+R(J2))*B1
      DM(J+1,I)=-(R(J1)+2.*R(J2))*B1
      DM(M,I)=(2.*VD+VT)*B2
      DM(M+1,I)=(VD+2.*VT)*B2
      DM(M+2,I)=DM(M,I)
  110 DM(M+3,I)=DM(M+1,I)
      DO 115 I=1,NCS6
      S=0.0
      DO 113 J=1,NSTR
  113 S=S+DM(I,J)*DM(I,J)
  115 DMT(I)=SQRT(S)
      DO 120 I=1,NSTR
      S=0.0
      DO 117 J=1,NSTR
  117 S=S+DEL(I,J)*DEL(I,J)
  120 DELT(I)=SQRT(S)
      CALL DERIV(B,-1,0,GDAC,KSTR,V1)
      RETURN
C     COMPUTE DERIVATIVES OF EIGEN-RESULTS
  122 DO 130 I=1,NBCM,2
      M=2*I
```

```
      B1=2.*Q/ELB
      DO 125 J=1,NSTR
      S1=2.*B1*RXT(M,J)+B1*RXT(M+1,J)
      S2=B1*RXT(M,J)+2.*B1*RXT(M+1,J)
      DO 125 K=1,NSTR
  125 G(K,J)=RXT(M,K)*S1+RXT(M+1,K)*S2
      CALL EIGEND(G,X,NSTR,I)
  130 CONTINUE
      IS=NBCM-2
      IF(NBCM.EQ.2) GO TO 141
      DO 140 I=2,IS,2
      K1=I/2
      K2=K1+1
      B1=2.*Q/EL(K1)
      J1=2*I-3
      J2=J1+4
      J3=J2-1
      J4=J3+4
      DO 133 J=1,NSTR
      S1=2.*B1*RXT(J1,J)+B1*RXT(J2,J)
      S2=2.*B1*RXT(J3,J)+B1*RXT(J4,J)
      S3=B1*RXT(J1,J)+2.*B1*RXT(J2,J)
      S4=B1*RXT(J3,J)+2.*B1*RXT(J4,J)
      DO 133 K=1,NSTR
  133 G(K,J)=RXT(J1,K)*S1+RXT(J3,K)*S2+RXT(J2,K)*S3+RXT(J4,K)*S4
      B1=3.*B1/EL(K1)
      DO 137 J=1,NSTR
      S=RXT(J1,J)+RXT(J2,J)+RXT(J3,J)+RXT(J4,J)
      G(J,K1)=G(J,K1)+B1*S
      G(J,K2)=G(J,K2)-B1*S
      G(K1,J)=G(K1,J)+B1*S
  137 G(K2,J)=G(K2,J)-B1*S
      B1=4.*B1/EL(K1)
      G(K1,K1)=G(K1,K1)+B1
      G(K2,K2)=G(K2,K2)+B1
      G(K1,K2)=G(K1,K2)-B1
      G(K2,K1)=G(K2,K1)-B1
      CALL EIGEND(G,X,NSTR,I)
  140 CONTINUE
  141 B1=2.*Q/EL(NSTR)
      DO 143 J=1,NSTR
      S1=2.*RXT(NDRF-3,J)*B1
      S2=2.*RXT(NDRF,J)*B1
      DO 143 K=1,NSTR
  143 G(K,J)=RXT(NDRF-3,K)*S1+RXT(NDRF,K)*S2
      B1=3.*B1/EL(NSTR)
      DO 145 J=1,NSTR
      S=RXT(NDRF-3,J)+RXT(NDRF,J)
      G(J,NSTR)=G(J,NSTR)+B1*S
  145 G(NSTR,J)=G(NSTR,J)+B1*S
      B1=4.*B1/EL(NSTR)
      G(NSTR,NSTR)=G(NSTR,NSTR)+B1
      CALL EIGEND(G,X,NSTR,NBCM)
      DO 150 I=1,NSTR
      J1=4*I-3
      J2=J1+1
      J3=J1+2
      J4=J1+3
      DO 147 J=1,NSTR
      S1=RXT(J1,J)-RXT(J2,J)
      S2=-S1
      S3=RXT(J3,J)-RXT(J4,J)
      S4=-S3
      DO 147 K=1,NSTR
  147 G(K,J)=RXT(J1,K)*S1+RXT(J2,K)*S2+RXT(J3,K)*S3+RXT(J4,K)*S4
      M=NBCM+I
      CALL EIGEND(G,X,NSTR,M)
  150 CONTINUE
  200 RETURN
      END



      SUBROUTINE EIGEND(G,A,NS,I)
      COMMON /DEIG/E(8),V(8,8),DE(8,24),DV(8,24,8)
      DIMENSION G(8,8),A(8,8)
      CALL MATMUL(A,G,V,NS,NS,NS,8,8,8)
      DO 10 J=1,NS
      DO 10 K=1,NS
      S=0.0
      DO 5 M=1,NS
    5 S=S+V(M,J)*A(M,K)
   10 G(J,K)=S
      DO 15 J=1,NS
      DE(J,I)=G(J,J)
      G(J,J)=0.0
      DO 15 K=1,NS
      IF(K.EQ.J) GO TO 15
      G(J,K)=G(J,K)/(E(K)-E(J))
   15 CONTINUE
      CALL MATMUL(A,V,G,NS,NS,NS,8,8,8)
      DO 20 J=1,NS
      DO 20 K=1,NS
   20 DV(K,I,J)=A(K,J)
      RETURN
      END



      SUBROUTINE MATMUL(A,B,C,L,M,N,LL,MM,NN)

C     THIS IS A MATRIX MULTIPLICATION ROUTINE

      DIMENSION A(LL,1),B(MM,1),C(NN,1)
      DO 10 I=1,L
      DO 10 J=1,N
      S=0.0
      DO 5 K=1,M
    5 S=S+B(I,K)*C(K,J)
   10 A(I,J)=S
      RETURN
      END



      SUBROUTINE NORMV(V,EM,NS)

C     THIS ROUTINE YIELDS VT*EM*V=I

      DIMENSION V(8,8),EM(8)
      DO 10 I=1,NS
      S=0.0
      DO 5 J=1,NS
    5 S=S+EM(J)*V(J,I)**2
      B=SQRT(S)
      DO 10 J=1,NS
   10 V(J,I)=V(J,I)/B
      RETURN
      END



      SUBROUTINE FDRIV(GRAD,IG,EN)
      COMMON /STDAT/NSTR,W(8),ELB,EL(8),Q,NBCM,NBCC,NCS4,NCDD,NCS6,NCSL,
     &NCD
      COMMON /RES/SM(48),DMT(48),DELT(8)
      COMMON /COST/CSMP,CSCP,CSW,CPT,COH,CI,FP(8),NAG,AG(10),DL
      COMMON /DESINF/NCC,MC(38),MUC(39),NSC,MS(16),COP,CC(2),CE(2),X(40)
      COMMON /CNSTINF/RCS,DA(8),BFS,RCD,DEQ(2),DUCA(16)
      DIMENSION GRAD(1),G(40)
      DATA KSWT/1/
      IF(IG.GE.0) GO TO 8
      IF(KSWT.EQ.0) GO TO 4
      DO 2 J=1,NCDD
    2 G(J)=0.0
      KSWT=0
    4 DO 6 J=1,NSTR
      CALL DERIV(X,J,3,0.,0,GRAD)
      FPP=FP(J)*EN
      DO 6 K=1,NBCC
    6 G(K)=G(K)+FPP*GRAD(K)
      RETURN
    8 IF(IG.GT.0) GO TO 12
      CALL DERIV(X,0,0,0.,0,GRAD)
      DO 10 J=1,NCDD
   10 GRAD(J)=GRAD(J)+CI*G(J)/2.
      KSWT=1
      IF(NCC.EQ.0) RETURN
      GO TO 200
   12 IF(IG.GT.NCSL) GO TO 50
      IF(IG.GT.NCS4) GO TO 22
      K=IG-1
      KM=MOD(K,4)
      MN=(K-KM)/2+1
      N=KM+1
      CALL DERIV(X,MN,1,0.,0,G)
      GO TO (13,14,13,14),N
   13 CT=1.
      GO TO 16
   14 CT=-1.
   16 DO 18 J=1,NBCC
   18 GRAD(J)=CT*G(J)
      NST=NBCC+1
      DO 20 J=NST,NCDD
   20 GRAD(J)=0.0
      GRAD(MN+NBCC)=-36.*RCS
      IF(NCC.EQ.0) RETURN
      GO TO 200
   22 IF(IG.GT.NCS4+NCS4) GO TO 40
      K=IG-NCS4-1
      KM=MOD(K,4)
      MN=(K-KM)/2+2
      Z=X(MN+NBCC)
      N=KM+1
      MN2=MN/2
      P=0.0
      DO 24 J=1,MN2
   24 P=P+W(J)/2.
      NN=(MN2-1)*4+NBCM
      PR=P*X(MN)/(1.1*Z*Z*36.)
      KK=1
      IF(PR.GE..15) KK=2
      GO TO (26,27,28,29),N
   26 CT=1.
      CALL DERIV(X,NN+1,1,0.,0,G)
      GO TO 30
   27 CT=-1.
      CALL DERIV(X,NN+1,1,0.,0,G)
      GO TO 30
   28 CT=1.
      CALL DERIV(X,NN+2,1,0.,0,G)
      GO TO 30
   29 CT=-1.
      CALL DERIV(X,NN+2,1,0.,0,G)
   30 DO 32 J=1,NBCC
   32 GRAD(J)=CT*G(J)
      NST=NBCC+1
      DO 34 J=NST,NCDD
   34 GRAD(J)=0.0
      GO TO (35,36),KK
   35 GRAD(MN+NBCC)=-36.*RCS
      GO TO 38
   36 GRAD(MN)=GRAD(MN)+1.1*P/Z*RCS
      GRAD(MN+NBCC)=-1.1*36.*(1.+PR)*RCS
   38 IF(NCC.EQ.0) RETURN
      GO TO 200
```

```
   40 IF(IG.GT.NCOD+NCS4) GO TO 46
      N=IG-NCS4-NCS4
      MN=2*N-1
      CT=ELB*ELB/(9.*Q*X(MN))
      CALL DERIV(X,MN,1,0.,0,G)
      DO 42 J=1,NBCC
   42 GRAD(J)=CT*G(J)
      NST=NBCC+1
      DO 44  J=NST,NCDD
   44 GRAD(J)=0.0
      GRAD(MN)=GRAD(MN)-ELB*ELB*(W(N)*ELB/384.+SM(MN)/8.)/(Q*X(MN)*X(MN)
     $)
      IF(NCC.EQ.0) RETURN
      GO TO 200
   46 PRINT 48
   48 FORMAT (/5X,*DERIVS. NOT AVAILABLE FOR BUCKLING CONSTRAINTS*)
      RETURN
   50 K=IG-NCSL
      L=1
      DMM=RCD
      IF(K.LE.NCS6) GO TO 52
      K=K-NCS6
      L=2
   52 IF(K.GT.NBCM) GO TO 64
      KM=MOD(K,2)
      N=(K+KM)/2
      MN=2*N-1
      IF(L.EQ.2) DMM=DUCA(MN)
      IF(KM.EQ.0) GO TO 54
      WL4=W(N)*ELB/4.
      IF(WL4.GT.DMT(MN)) GO TO 53
      CCS=1.
      CCD=1.
      GO TO 56
   53 CCS=1.
      CCD=DMT(MN)/WL4
      GO TO 56
   54 CCS=-1.
      CCD=1.
   56 CALL DERIV(X,MN,1,0.,0,G)
      DO 58 J=1,NBCC
   58 GRAD(J)=CCS*G(J)
      CALL DERIV(X,MN,2,0.,0,G)
      DO 60  J=1,NBCC
   60 GRAD(J)=GRAD(J)+CCD*G(J)
      NST=NBCC+1
      DO 62  J=NST,NCDD
   62 GRAD(J)=0.0
      GRAD(MN+NBCC)=-36.*DMM
      IF(NCC.EQ.0) RETURN
      GO TO 200
   64 KK=K-NBCM-1
      KM=MOD(KK,4)
      MN=(KK-KM)/2+2
      Z=X(MN+NBCC)
      IF(L.EQ.2) DMM=DUCA(MN)
      N=KM+1
      MN2=MN/2
      P=0.0
      DO 66  J=1,MN2
   66 P=P+W(J)/2.
      PR=P*X(MN)/(1.1*7*7*36.)
      KK=1
      IF(PR.GE..15) KK=2
      GO TO (67,68,69,70),N
   67 CT=1.
      KM=K
      GO TO 71
   68 CT=-1.
      KM=K-1
      GO TO 71
   69 CT=1.
      KM=K-1
      GO TO 71
   70 CT=-1.
      KM=K-2
   71 CALL DERIV(X,KM,1,0.,0,G)
      DO 72 J=1,NBCC
   72 GRAD(J)=CT*G(J)
      CALL DERIV(X,KM,2,0.,0,G)
      DO 74  J=1,NBCC
   74 GRAD(J)=GRAD(J)+G(J)
      NST=NBCC+1
      DO 76  J=NST,NCDD
   76 GRAD(J)=0.0
      GO TO (77,78),KK
   77 GRAD(MN+NBCC)=-36.*DMM
      GO TO 80
   78 GRAD(MN)=GRAD(MN)+1.1*P/Z*DMM
      GRAD(MN+NBCC)=-1.18*36.*(1.+PR)*DMM
   80 IF(NCC.EQ.0) RETURN
  200 IF(NSC.EQ.0) GO TO 208
      DO 206 J=1,NSC
      K=MS(J)
      L=K+NBCC
      IF(MOD(K,2)) 202,204,202
  202 PD=.61*X(K)**(-.287)
      GO TO 206
  204 KK=1
      IF(X(K).GE.CDP) KK=2
      PD=CC(KK)*CE(KK)*X(K)**(CE(KK)-1.)
  206 GRAD(K)=GRAD(K)+GRAD(L)*PD
  208 DO 210 J=1,NCD
      K=MUC(J)
  210 GRAD(J)=GRAD(K)
      RETURN
      END
```

```
      SUBROUTINE DERIV(B,IG,ID,GDAC,KSTR,G)
      COMMON /STDAT/NSTR,W(8),ELB,EL(8),Q,NBCM,NBCC,NORF,NCDD,NCS6,NCSL,
     $NCD
      COMMON /DEIG/E(8),V(8,8),DE(8,24),DV(8,24,8)
      COMMON /ENDRV/Y(8),EM(8),DS(36),SR(32,32),RD(32,8),RXT(32,8),
     $DM(48,8),DEL(8,8)
      COMMON /RES/SM(48),DMT(48),DELT(8)
      COMMON /COST/CSMP,CSCP,CSW,CPT,COH,CI,FP(8),NAG,AG(10),DL
      DIMENSION B(1),G(1),DX(8,24,8)
      IF(IG.GT.0) GO TO 25
      IF(IG.LT.0) GO TO 6
      DO 1 I=1,NBCM,2
      EI=B(I)
      EI3=EI**3
      Z=B(I+NBCC)
      Z2=Z**2
      Z4=Z2*Z2
      A=1.1*Z2/EI
      DA=-A/EI
      S=.002833*ELB*DA*(CSMP+.72*A**(-.21))
      S=S-.86*CSCP*.002833*Z4/EI3
      S=S-.002833*CSW*Z2*(.73+.0052*Z2/EI+.096*Z/SQRT(EI))/EI**2
      S=S+2.*CPT*ELB*(19.65*EI**6-22.85*EI3*Z4+1.48*Z**8)/(2.09*EI3-.81*
     $Z4)**2/Z
    1 G(I)=COH*S
      DO 2 I=2,NBCM,2
      EI=B(I)
      EI3=EI**3
      Z=B(I+NBCC)
      Z2=Z*Z
      Z4=Z2*Z2
      A=1.1*Z2/EI
      DA=-A/EI
      ELC=EL(I/2)
      S=.002833*ELC*DA*(CSMP+.72*A**(-.21))
      S=S+2.*CPT*ELC*(19.65*EI**6-22.85*EI3*Z4+1.48*Z**8)/(2.09*EI3-.81*
     $Z4)**2/Z
    2 G(I)=COH*S
      IB=NBCM+1
      DO 3 I=IB,NBCC
      Z2=B(2*I-NSTR-1)**2
      S=-.72*CSCP*.002833*Z2*10800./D(I)**2
    3 G(I)=COH*S
      IB=NBCC+1
      DO 4 I=IB,NCDD,2
      EI=B(I-NBCC)
      EI3=EI**3
      Z=B(I)
      Z2=Z*Z
      Z4=Z2*Z2
      EK=B((I-NBCC+1)/2+NBCM)
      A=1.1*Z2/EI
      DA=2.2*Z/EI
      S=.002833*ELB*DA*(CSMP+.72*A**(-.21))
      S=S+.72*CSCP*.002833*(2.4*Z**3/EI**2+21600.*Z/EK)
      S=S+.002833*CSW*Z*(1.46+.0104*Z2/EI+.192*Z/SQRT(EI))/EI
      S=S+2.*CPT*ELB*EI*(-19.65*EI**6+26.65*EI3*Z4-1.48*Z**8)/(2.09*EI3-
     $.81*Z4)**2/Z2
    4 G(I)=COH*S
      IB=IB+1
      DO 5 I=IB,NCDD,2
      EI=B(I-NBCC)
      EI3=EI**3
      Z=B(I)
      Z2=Z*Z
      Z4=Z2*Z2
      ELC=EL((I-NBCC)/2)
      A=1.1*Z2/EI
      DA=2.2*Z/EI
      S=.002833*ELC*DA*(CSMP+.72*A**(-.21))
      S=S+2.*CPT*ELC*EI*(-19.65*EI**6+26.65*EI3*Z4-1.48*Z**8)/(2.09*EI3-
     $.81*Z4)**2/Z2
    5 G(I)=COH*S
      RETURN
    6 DO 22 I=1,NSTR
      FQ=SQRT(E(I))
      IF(KSTR.EQ.1) GO TO 12
      IF(FQ.GE.1.81) GO TO 8
      SD=36.*GDAC*1.4
      DSD=0.0
      GO TO 18
    8 IF(FQ.GT.11.) GO TO 10
      SD=48.*GDAC*1.9/FQ
      DSD=-SD/FQ
      GO TO 18
   10 SD=386.*GDAC*2.6/(FQ*FQ)
      DSD=-2.*SD/FQ
      GO TO 18
   12 IF(FQ.GE.1.58) GO TO 14
      SD=36.*GDAC*1.1
      DSD=0.0
      GO TO 18
   14 IF(FQ.GT.14.01) GO TO 16
      SD=48.*GDAC*1.3/FQ
      DSD=-SD/FQ
      GO TO 18
   16 SD=386.*GDAC*1.5*1.51/(FQ*FQ)
      DSD=-2.*SD/FQ
   18 S1=Y(I)*DSD/(2.*FQ*SD)
      DO 22 J=1,NBCC
      S=0.0
      DO 20 K=1,NSTR
   20 S=S+DV(K,J,I)*EM(K)
      S=S*SD+S1*DE(I,J)
      DO 22 K=1,NSTR
   22 DX(K,J,I)=DV(K,J,I)*Y(I)+V(K,I)*S
      RETURN
   25 IF(ID.EQ.3) GO TO 110
      IF(IG.GT.NBCM) GO TO 27
      M=IG
      J=2*IG
      K=J+1
```

IX

```
      L=1
      B2=2.*Q/ELB
      GO TO 29
   27 L=MOD(IG,2)
      M=(IG+L-NBCM)/2+1
      MX=M/2
      B2=2.*Q/EL(MX)
      J=2*M-3
      K=J+4
      IF(M.EQ.NBCM) K=J
   29 IF(ID.EQ.2) GO TO 52
      B1=2.*Q/ELB
      DO 34 I=1,NBCM,2
      J1=2*I
      J2=J1+1
      S1=2.*B1*DS(J1)+B1*DS(J2)
      S2=B1*DS(J1)+2.*B1*DS(J2)
      S3=-S1*SR(J,J1)-S2*SR(J,J2)
      S4=-S1*SR(K,J1)-S2*SR(K,J2)
      IF(M.EQ.NBCM) S4=0.0
      S2=0.0
      IF(L) 31,32,31
   31 S1=2.*B2*B(M)*S3+B2*B(M)*S4
      IF(I.EQ.M) S2=2.*B2*DS(J)+B2*DS(K)
      GO TO 34
   32 S1=B2*B(M)*S3+2.*B2*B(M)*S4
      IF(I.EQ.M) S2=B2*DS(J)+2.*B2*DS(K)
   34 G(I)=S1+S2
      NST=NBCM-2
      IF(NBCM.EQ.2) GO TO 41
      DO 39 I=2,NST,2
      I2=I/2
      B1=2.*Q/EL(I2)
      J1=2*I-3
      J2=J1+4
      J3=J2-1
      J4=J3+4
      S1=2.*B1*DS(J1)+B1*DS(J2)
      S2=2.*B1*DS(J3)+B1*DS(J4)
      S3=B1*DS(J1)+2.*B1*DS(J2)
      S4=B1*DS(J3)+2.*B1*DS(J4)
      S=-S1*SR(J,J1)-S2*SR(J,J3)-S3*SR(J,J2)-S4*SR(J,J4)
      T=-S1*SR(K,J1)-S2*SR(K,J3)-S3*SR(K,J2)-S4*SR(K,J4)
      IF(M.EQ.NBCM) T=0.C
      S2=0.0
      IF(L) 36,37,36
   36 S1=2.*B2*B(M)*S+B2*B(M)*T
      IF(I.EQ.M) S2=2.*B2*DS(J)+B2*DS(K)
      GO TO 39
   37 S1=B2*B(M)*S+2.*B2*B(M)*T
      IF(I.EQ.M) S2=B2*DS(J)+2.*B2*DS(K)
   39 G(I)=S1+S2
   41 I=NBCM
      B1=2.*Q/EL(NSTR)
      NDW=NDRF-3
      S1=2.*B1*DS(NDW)
      S2=2.*B1*DS(NDRF)
      S=-S1*SR(J,NDW)-S2*SR(J,NDRF)
      T=-S1*SR(K,NDW)-S2*SR(K,NDRF)
      IF(M.EQ.NBCM) T=0.0
      S2=0.0
      IF(L) 43,44,43
   43 S1=2.*B2*B(M)*S+B2*B(M)*T
      IF(I.EQ.M) S2=2.*B2*DS(J)
      GO TO 46
   44 S1=B2*B(M)*S+2.*B2*B(M)*T
      IF(I.EQ.M) S2=B2*DS(J)
   46 G(I)=S1+S2
      IB=NBCM+1
      DO 50 I=IB,NBCC
      J1=4*(I-NBCM)-3
      J2=J1+1
      J3=J1+2
      J4=J1+3
      S1=DS(J1)-DS(J2)
      S2=-S1
      S3=DS(J3)-DS(J4)
      S4=-S3
      S=-S1*SR(J,J1)-S2*SR(J,J2)-S3*SR(J,J3)-S4*SR(J,J4)
      T=-S1*SR(K,J1)-S2*SR(K,J2)-S3*SR(K,J3)-S4*SR(K,J4)
      IF(M.EQ.NBCM) T=0.0
      IF(L) 48,49,48
   48 S1=2.*B2*B(M)*S+B2*B(M)*T
      GO TO 50
   49 S1=B2*B(M)*S+2.*B2*B(M)*T
   50 G(I)=S1
      RETURN
C     COMPUTE DERIVATIVES OF DYNAMIC MOMENTS
   52 B1=2.*Q/ELB
      DO 69 I=1,NBCM,2
      J1=2*I
      J2=J1+1
      SP=0.0
      DO 67 MD=1,NSTR
      S1=2.*B1*RD(J1,MD)+B1*RD(J2,MD)
      S2=B1*RD(J1,MD)+2.*B1*RD(J2,MD)
      S3=S1*SR(J,J1)+S2*SR(J,J2)
      S4=S1*SR(K,J1)+S2*SR(K,J2)
      S1=0.0
      S2=0.0
      DO 54 NN=1,NSTR
      S1=S1+RXT(J,NN)*DX(NN,I,MD)
   54 S2=S2+RXT(K,NN)*DX(NN,I,MD)
      S1=S3-S1
      S2=S4-S2
      IF(IG.LE.NBCM) GO TO 60
      IF(M.NE.NBCM) GO TO 56
      S2=0.0
      S3=-DX(MX,I,MD)/EL(MX)
      GO TO 58
   56 S3=(DX(MX+1,I,MD)-DX(MX,I,MD))/EL(MX)
   58 S1=S1+S3
      S2=S2+S3
   60 S4=0.0
      IF(L) 62,63,62
   62 S3=2.*B2*B(M)*S1+B2*B(M)*S2
      IF(I.EQ.M) S4=-2.*B2*RD(J,MD)-B2*RD(K,MD)
      GO TO 65
   63 S3=B2*B(M)*S1+2.*B2*B(M)*S2
      IF(I.EQ.M) S4=-B2*RD(J,MD)-2.*B2*RD(K,MD)
   65 S=S3+S4
   67 SP=SP+S*DM(IG,MD)
   69 G(I)=SP/DMT(IG)
      IF(NBCM.EQ.2) GO TO 88
      NST=NBCM-2
      DO 86 I=2,NST,2
      I2=I/2
      B1=2.*Q/EL(I2)
      B3=3.*B1/EL(I2)
      J1=2*I-3
      J2=J1+4
      J3=J2-1
      J4=J3+4
      SP=0.0
      DO 84 MD=1,NSTR
      S1=2.*B1*RD(J1,MD)+B1*RD(J2,MD)
      S2=2.*B1*RD(J3,MD)+B1*RD(J4,MD)
      S3=B1*RD(J1,MD)+2.*B1*RD(J2,MD)
      S4=B1*RD(J3,MD)+2.*B1*RD(J4,MD)
      S=S1*SR(J,J1)+S2*SR(J,J3)+S3*SR(J,J2)+S4*SR(J,J4)
      T=S1*SR(K,J1)+S2*SR(K,J3)+S3*SR(K,J2)+S4*SR(K,J4)
      S1=0.0
      S2=0.0
      DO 71 NN=1,NSTR
      S1=S1+RXT(J,NN)*DX(NN,I,MD)
   71 S2=S2+RXT(K,NN)*DX(NN,I,MD)
      S=S-S1
      T=T-S2
      S1=-DEL(I2,MD)*B3
      S3=S1*(SR(J,J1)+SR(J,J3)+SR(J,J2)+SR(J,J4))
      S4=S1*(SR(K,J1)+SR(K,J3)+SR(K,J2)+SR(K,J4))
      S=S-S3
      T=T-S4
      IF(IG.LE.NBCM) GO TO 77
      IF(M.NE.NBCM) GO TO 73
      S3=-DX(MX,I,MD)/EL(MX)
      T=0.0
      GO TO 75
   73 S3=(DX(MX+1,I,MD)-DX(MX,I,MD))/EL(MX)
   75 S=S+S3
      T=T+S3
   77 S4=0.0
      IF(L) 79,80,79
   79 S3=2.*B2*B(M)*S+B2*B(M)*T
      IF(I.NE.M) GO TO 82
      S2=-DEL(MX,MD)/EL(MX)
      S=-RD(J,MD)+S2
      T=-RD(K,MD)+S2
      S4=2.*B2*S+B2*T
      GO TO 82
   80 S3=B2*B(M)*S+2.*B2*B(M)*T
      IF(I.NE.M) GO TO 82
      S2=-DEL(MX,MD)/EL(MX)
      S=-RD(J,MD)+S2
      T=-RD(K,MD)+S2
      S4=B2*S+2.*B2*T
   82 S=S3+S4
   84 SP=SP+S*DM(IG,MD)
   86 G(I)=SP/DMT(IG)
   88 I=NBCM
      B1=2.*Q/EL(NSTR)
      NDW=NDRF-3
      B3=3.*B1/EL(NSTR)
      SP=0.0
      DO 94 MD=1,NSTR
      S1=2.*B1*RD(NDW,MD)
      S2=2.*B1*RD(NDRF,MD)
      S=S1*SR(J,NDW)+S2*SR(J,NDRF)
      T=S1*SR(K,NDW)+S2*SR(K,NDRF)
      S1=0.0
      S2=0.0
      DO 90 NN=1,NSTR
      S1=S1+RXT(J,NN)*DX(NN,I,MD)
   90 S2=S2+RXT(K,NN)*DX(NN,I,MD)
      S=S-S1
      T=T-S2
      S1=-DEL(NSTR,MD)*B3
      S3=S1*(SR(J,NDW)+SR(J,NDRF))
      S4=S1*(SR(K,NDW)+SR(K,NDRF))
      S=S-S3
      T=T-S4
      IF(IG.LE.NBCM) GO TO 186
      IF(M.NE.NBCM) GO TO 182
      S3=-DX(MX,I,MD)/EL(MX)
      T=0.0
      GO TO 184
  182 S3=(DX(MX+1,I,MD)-DX(MX,I,MD))/EL(MX)
  184 S=S+S3
      T=T+S3
  186 S4=0.0
      IF(L) 189,190,189
  189 S3=2.*B2*B(M)*S+B2*B(M)*T
      IF(I.NE.M) GO TO 92
      S2=-DEL(MX,MD)/EL(MX)
      S=-RD(J,MD)+S2
      T=S2
      S4=2.*B2*S+B2*T
      GO TO 92
  190 S3=B2*B(M)*S+2.*B2*B(M)*T
      IF(I.NE.M) GO TO 92
      S2=-DEL(MX,MD)/EL(MX)
      S=-RD(J,MD)+S2
      T=S2
      S4=B2*S+2.*B2*T
```

X

```
   92 S=S3+S4
   94 SP=SP+S*DM(IG,MD)
      G(I)=SP/DMT(IG)
      IB=NBCM+1
      DO 109 I=IB,NBCC
      J1=4*(I-NBCM)-3
      J2=J1+1
      J3=J1+2
      J4=J1+3
      SP=0.0
      DO 106 MD=1,NSTR
      S1=RD(J1,MD)-RD(J2,MD)
      S2=-S1
      S3=RD(J3,MD)-RD(J4,MD)
      S4=-S3
      S=S1*SR(J,J1)+S2*SR(J,J2)+S3*SR(J,J3)+S4*SR(J,J4)
      T=S1*SR(K,J1)+S2*SR(K,J2)+S3*SR(K,J3)+S4*SR(K,J4)
      S1=0.0
      S2=0.0
      DO 96 NN=1,NSTR
      S1=S1+RXT(J,NN)*DX(NN,I,MD)
   96 S2=S2+RXT(K,NN)*DX(NN,I,MD)
      S=S-S1
      T=T-S2
      IF(IG.LE.NBCM) GO TO 102
      IF(M.NE.NBCM) GO TO 98
      S3=-DX(MX,I,MD)/EL(MX)
      T=0.0
      GO TO 100
   98 S3=(DX(MX+1,I,MD)-DX(MX,I,MD))/EL(MX)
  100 S=S+S3
      T=T+S3
  102 IF(L) 104,105,104
  104 S3=2.*B2*B(M)*S+B2*B(M)*T
      GO TO 106
  105 S3=B2*B(M)*S+2.*B2*B(M)*T
  106 SP=SP+S3*DM(IG,MD)
  108 G(I)=SP/DMT(IG)
      RETURN
  110 IF(IG.EQ.NSTR) GO TO 116
      DO 114 I=1,NBCC
      S=0.0
      DO 112 J=1,NSTR
  112 S=S+(DX(IG,I,J)-DX(IG+1,I,J))*DEL(IG,J)
  114 G(I)=S/DELT(IG)
      RETURN
  116 DO 120 I=1,NBCC
      S=0.0
      DO 118 J=1,NSTR
  118 S=S+DX(IG,I,J)*DEL(IG,J)
  120 G(I)=S/DELT(IG)
      RETURN
      END


      SUBROUTINE HORW (N,NM,M,G,E,V,A,B,P,W,Q,INT)
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C     SUBROUTINE TO COMPUTE EIGENVALUES AND EIGENVECTORS OF A
C     SYMMETRIC MATRIX STORED AS A TWO-DIMENSIONAL ARRAY
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C     CARLOS A. FELIPPA, FEB. 1967.
C
C
C  INPUTS
C
C       N     MATRIX ORDER, MUST NOT EXCEED NM.
C
C       NM    DIMENSION OF INPUT MATRIX G IN THE CALLING PROGRAM.
C
C       M     NVEC = IABS(M)  IS THE NUMBER OF EIGENVECTORS DESIRED
C             (0 TO N). ITS SIGN SPECIFIES THE ORDERING OF THE
C             EIGENVALUES   E(1) ..... E(N)  AS FOLLOWS
C               IF M LT 0 OR -0, BY INCREASING ALGEBRAIC VALUE
C               IF M GT 0 OR +0, BY DECREASING ALGEBRAIC VALUE.
C             CALCULATED EIGENVECTORS (IF ANY) WILL CORRESPOND TO
C             E(1), E(2) ... E(NVEC)
C
C       G     INPUT SYMMETRIC SQUARE MATRIX (RETURNS UNALTERED).
C
C
C  OUTPUTS
C
C       E     VECTOR OF EIGENVALUES, ARRANGED AS EXPLAINED ABOVE.
C
C       V     NORMALIZED EIGENVECTORS, STORED AS COLUMNS OF V.
C             IF NVEC=0, V MAY BE A DUMMY VARIABLE.
C
C       A     DIAGONAL OF REDUCED TRIDIAGONAL FORM.
C
C       B     FIRST OFF-DIAGONAL OF REDUCED TRIDIAGONAL FORM.
C
C  WORKING SPACE
C
C       P,W,Q,INT     WORKING VECTORS OF LENGTH AT LEAST N,N+1,N AND N
C                     RESPECTIVELY. IF NVEC=0, Q AND INT MAY BE
C                     DUMMY VARIABLES.
C
C
C     MINIMUM DIMENSIONS IN THE CALLING PROGRAM SHOULD BE
C     G(NM,N), E(N), V(NM,NVEC), A(N), B(N), P(N), W(N+1), Q(N), INT(N)
C      BUT V,Q AND INT CAN BE DUMMIES IF NVEC=0 (NO EIGENVECTORS).
C
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C     THE FOLLOWING PARAMETERS ARE MACHINE-DEPENDENT AND SHOULD
C     BE PRE-SET AS FOLLOWS
C
C       PRECS = 10.**(-NDIG)     WHERE NDIG IS THE NUMBER OF SIGNIFICANT
C                  DECIMAL DIGITS CARRIED OUT BY THE MACHINE IN FLOATING
C                  POINT ARITHMETIC.
C       ILIM =  TO BE CHOSEN SO THAT BASE**(ILIM+4) IS OF THE ORDER
C                  (BUT DOES NOT EXCEED) THE MACHINE OVERFLOW LIMIT.
C       HOV  =  BASE**(ILIM/2)
C     THIS VERSION IS FOR THE CDC 6400  (NDIG=15, BASE=2., ILIM=1000)
C
C       BASE =  THE BASE NUMBER OF THE MACHINE, IN FLOATING POINT.
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
      DIMENSION  G(NM,1), E(1), V(NM,1), A(1), B(1), P(1), W(1), Q(1)
      REAL LAMBDA
      LOGICAL INT(1)
      IF (N.LE.0.OR.N.GT.NM)  GO TO 1000
      PRECS = 1.0E-15
      BASE = 2.0
      ILIM = 1000
      HOV = BASE**500
      B(1) = 0.
      SQRT2 = SQRT(2.)
      N1 = N - 1
      DO 100  I = 1,N
  100 E(I) = G(I,I)
      IF  (N-2)  900,280,110
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C     TRI-DIAGONALIZE MATRIX G BY HOUSEHOLDER#S PROCEDURE
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
  110 DO 250  K = 2,N1
      K1 = K - 1
      KJ = K + 1
      Y = G(K,K1)
      SUM = 0.0
      DO 120  I = KJ,N
  120 SUM = SUM + G(I,K1)**2
      IF (SUM.EQ.0.)  GO TO 230
      S = SQRT(SUM+Y**2)
      B(K) = SIGN(S,-Y)
      W(K) = SQRT(1.+ABS(Y)/S)
      X = SIGN(1./(S*W(K)),Y)
      DO 150  I = K,N
      IF (I.GT.K)  W(I) = X*G(I,K1)
      P(I) = 0.
  150 G(I,K1) = W(I)
      DO 180  I = K,N
      Y = W(I)
      IF (Y.EQ.0.)  GO TO 180
      I1 = I + 1
      DO 160  J = K,I
  160 P(J) = P(J) + Y*G(I,J)
      IF (I1.GT.N)  GO TO 180
      DO 170  J = I1,N
  170 P(J) = P(J) + Y*G(J,I)
  180 CONTINUE
  190 X = 0.
      DO 200  J = K,N
  200 X = X + W(J)*P(J)
      X = 0.5*X
      DO 210  J = K,N
  210 P(J) = X*W(J) - P(J)
      DO 220  J = K,N
      DO 220  I = J,N
  220 G(I,J) = G(I,J) + P(I)*W(J) + P(J)*W(I)
      GO TO 250
  230 G(K,K1) = SQRT2
      B(K) = -Y
      DO 240  I = KJ,N
  240 G(I,K) = -G(I,K)
  250 CONTINUE
  280 DO 290  I = 1,N
      A(I) = G(I,I)
  290 G(I,I) = E(I)
      B(N) = G(N,N1)
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C     GET EIGENVALUES OF TRIDIAGONAL FORM BY KAHAN-VARAH Q-R METHOD
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
      TOL = PRECS/(10.*FLOAT(N))
      BMAX = 0.
      TMAX = 0.
      W(N+1) = 0.
      DO 300  I = 1,N
      BMAX = AMAX1(BMAX,ABS(B(I)))
  300 TMAX = AMAX1(BMAX,ABS(A(I)),TMAX)
      SCALE = 1.0
      IF (BMAX.EQ.0.)  GO TO 520
      DO 310  I = 1,ILIM
      IF (SCALE*TMAX.GT.HOV)  GO TO 320
  310 SCALE = SCALE*BASE
  320 DO 330  I = 1,N
      E(I) = A(I)*SCALE
  330 W(I) = (B(I)*SCALE)**2
      DELTA = TMAX*SCALE*TOL
      EPS = DELTA**2
      K=N
  350 L = K
      IF (L.LE.0)  GO TO 460
      L1 = L - 1
      DO 360  I = 1,L
      K1 = K
      K = K - 1
      IF (W(K1).LT.EPS)  GO TO 380
  360 CONTINUE
  380 IF (K1.NE.L)  GO TO 400
      W(L) = 0.
      GO TO 350
  400 T = E(L) - E(L1)
      X = W(L)
      Y = 0.5*T
```

XI

```
      S = SQRT(X)
      IF (ABS(T).GT.DELTA)  S = (X/Y)/(1.+SQRT(1.+X/Y**2))
      E1 = E(L) + S
      E2 = E(L1)- S
      IF (K1.NE.L1)  GO TO 430
      E(L)  = E1
      E(L1) = E2
      W(L1) = 0.
      GO TO 350
  430 LAMBDA = E1
      IF (ABS(T).LT.DELTA.AND.ABS(E2).LT.ABS(E1))  LAMBDA = E2
      S = 0.
      C = 1.
      GG = E(K1)-LAMBDA
      GO TO 450
  440 C = F/T
      S = X/T
      X = GG
      GG = C*(E(K1)-LAMBDA) - S*X
      E(K) = (X-GG) + E(K1)
  450 IF (ABS(GG).LT.DELTA) GG = GG + SIGN(C*DELTA,GG)
      F = GG**2/C
      K = K1
      K1 = K + 1
      X = W(K1)
      T = X + F
      V(K) = S*T
      IF (K.LT.L)  GO TO 440
      E(K) = GG + LAMBDA
      GO TO 350
  460 DO 470  I = 1,N
  470 E(I) = E(I)/SCALE
      Y =  ISIGN(1,M)
      DO 500  L = 1,N1
      K = N - L
      DO 500  I = 1,K
      IF (Y*(E(I)-E(I+1)).GT.0.)  GO TO 500
      X = E(I)
      E(I) = E(I+1)
      E(I+1) = X
  500 CONTINUE
  520 IF (M.EQ.0)  GO TO 860
C
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C     COMPUTE EIGENVECTORS BY INVERSE ITERATION
C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C
      NVEC = IABS(M)
      IF (NVEC.GT.N)  NVEC = N
      F = SCALE/HPV
      IF (BMAX*F.LT.PRECS)  GO TO 830
      DO 530  I = 1,N
      A(I) = A(I)*F
  530 B(I) = B(I)*F
      SEP = 25.*TMAX*PRECS
      X1 = 0.
      X2 = SQRT2
      DO 800  NV = 1,NVEC
      IF ( NV .LE. 1 ) GO TO 545
      IF ( ABS(E(NV)-E(NV-1)) .LT. SEP ) GO TO 545
  545 DO 540 I=1,N
  540 W(I) = 1.0
      GO TO 570
  550 DO 560  I = 1,N
      X = AMOD(X1+X2,2.0)
      X1 = X2
      X2 = X
  560 W(I) = X - 1.0
  570 EV = E(NV)*F
      X = A(1) - EV
      Y = B(2)
      J = N1
      DO 600  I = 1,N1
      C = A(I+1) - EV
      S = B(I+1)
      IF (ABS(X).GE.ABS(S))  GO TO 580
      P(I) = S
      Q(I) = C
      INT(I) = .TRUE.
      Z = -X/S
      X = Y + Z*C
      IF (I.LT.N1)  Y = Z*B(I+2)
      GO TO 600
  580 IF (ABS(X).LT.TOL)  X = TOL
      P(I) = X
      Q(I) = Y
      INT(I) = .FALSE.
      Z = -S/X
      X = C + Z*Y
      Y = B(I+2)
  600 V(I,NV) = Z
      IF (ABS(X).LT.TOL)  X = TOL
      NITER = 0
  620 NITER = NITER + 1
      W(N) = W(N)/X
      SUM = W(N)**2
      DO 640  L = 1,N1
      I = N - L
      Y = W(I) - Q(I)*W(I+1)
      IF ( I .GE. N1 ) GO TO 630
      IF (INT(I))  Y = Y - B(I+2)*W(I+2)
  630 W(I)=Y/P(I)
  640 SUM = SUM + W(I)**2
      S = SQRT(SUM)
      DO 660  I = 1,N
  660 W(I) = W(I)/S
      IF (NITER.GE.2)  GO TO 760
      DO 700  I = 1,N1
      Z = V(I,NV)
      IF (INT(I))  GO TO 680
      W(I+1) = W(I+1) + Z*W(I)
      GO TO 700
  680 Y = W(I)
      W(I) = W(I+1)
      W(I+1) = Y + Z*W(I)
  700 CONTINUE
      GO TO 620
  730 L = J
      J = J - 1
      X = 0.
      DO 740  I = L,N
  740 X = X + G(I,J)*W(I)
      DO 750  I = L,N
  750 W(I) = W(I) - X*G(I,J)
  760 IF (J.GT.1)  GO TO 730
      DO 800  I = 1,N
  800 V(I,NV) = W(I)
      DO 820  I = 1,N
      A(I) = A(I)/F
  820 B(I) = B(I)/F
      GO TO 860
  830 DO 850  NV = 1,NVEC
      DO 840  I = 1,N
  840 V(I,NV) = 0.
  850 V(NV,NV) = 1.0
  860 DO 880  I = 2,N
      K = I - 1
      DO 880  J = 1,K
  880 G(I,J) = G(J,I)
      GO TO 1000
  900 V(1,1) = 1.0
      A(1) = E(1)
 1000 RETURN
      END
```

```
* * * * * * * * * * * * * * * *
* TWO STORY EXAMPLE DATA DECK *
* * * * * * * * * * * * * * * *


    2
144.      180.
240.
500.      500.      2000.     1000.     90000000009000000000071.14     80.08
191.89    155.39
60.       100.
0.15      0.45
13.       26.       0.21      449.80    0.00066   0.1       0.1
.1        50.          4
0.0       0.065     0.309     0.5
0.035     0.03
    6    4
    5    6    7    8    9   10
    1    2    3    4
429.      2.47      0.21      0.553     0.9564
.6        .85       .6
2.        2.
6.        1.        6.        1.
500.      2.        10000.       300.      2.       3000.
20.       50.       0.5       0.001     0.003     20.
   20    1    5   30
    2   22   39   43
    1
```

XII

```
* * * * * * * * * * * * *
* OUTPUT FROM EXAMPLE *
* * * * * * * * * * * * *
```

```
   ----- NERD INPUT DATA -----

NUMBER OF STORIES---  2

STORY HEIGHTS (TOP DOWN)---    144.00    180.00

BAY WIDTH---    240.00

MEMBER MOMENTS OF INERTIA (TOP DOWN)---     500.00     500.00    2000.00    1000.00

CONNECTION STIFFNESSES (TOP DOWN)---9000000000.9000000000.

MEMBER PLASTIC SECTION MODULI (TOP DOWN)---    71.14    80.08   191.89   155.39

BEAM LOADING (TOP DOWN)---     60.00    100.00

DESIGN EARTHQUAKES---      .150      .450

CONSTRUCTION COSTS:

   MEMBER STEEL---   13.00

   CONNECTION STEEL---   26.00

   STEEL TRANSPORTATION---     .21

   WELD METAL---  449.80

   STRUCTURAL PAINTING--- .000660

   OVERHEAD---      .10

   PROFIT---      .10

DAMAGE COSTS:

   DOWN TIME COST MULTIPLIER---     .10

   DESIGN LIFE---   50.00

   COST ESTIMATION EARTHQUAKES---   0.        .065     .309     .500

   FLOOR PARTICIPATION FACTORS (TOP DOWN)---     .0350     .0300

THERE ARE  6 CONSTRAINED COMPONENTS IN THE DESIGN VECTOR:   5   6   7   8   9  10

THERE ARE  4 MEMBERS WITH ASSIGNED PLASTIC SECTION MODULI:   1   2   3   4

ASSIGNED COLUMN PLASTIC SECTION MODULUS:
```

DIVIDING PT.--- 429.00

COEFFICIENT--- 2.47 .21

EXPONENT--- .553 .956

STATIC LOADING REDUCTION COEFFICIENT--- .60

DYNAMIC LOADING REDUCTION COEFFICIENT--- .85

BUCKLING REDUCTION COEFFICIENT--- .60

FLOOR DISPLACEMENT ALLOWABLES (TOP DOWN)--- 2.000 2.000

ALLOWABLE DUCTILITY (TOP DOWN)--- 6.0 1.0 6.0 1.0

INITIAL COST TIMES (ONE PLUS DOWN TIME MULTIPLIER)--- 5699.45

UNCONSTRAINED MINIMIZATION PARAMETERS:

BEAM STEP SIZE UPPER LIMIT--- 500.0

BEAM STEP SIZE LOWER LIMIT--- 2.0

BEAM STEP MULTIPLIER--- 1.00E+04

COLUMN STEP SIZE UPPER LIMIT--- 300.0

COLUMN STEP SIZE LOWER LIMIT--- 2.0

COLUMN STEP MULTIPLIER--- 3.00E+03

ALGORITHM ACCURACY BOUNDS:

UNCONSTRAINED MINIMIZATION DESIGN VARIABLE TOLERANCE--- 20.0

MOMENT CONSTRAINTS TOLERANCE--- 50.0

DISPLACEMENT CONSTRAINTS TOLERANCE--- .500

ALGORITHM TERMINATION VALUE FOR BEAMS--- 1.00E-03

ALGORITHM TERMINATION VALUE FOR COLUMNS--- 3.00E-03

INITIAL STEP SIZE FOR GRADIENT PROJECTION--- 20.0

ALGORITHM ITERATION LIMITS:

UNCONSTRAINED MINIMUM SEARCH--- 20

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```
   PHASE ONE RESTARTS---    1

   CONSTRAINT FUNCTION SEARCH---    5

   CONSTRAINED MINIMUM SEARCH---   30

PRIMARY CONSTRAINTS:    2   22   39   43

MEMBER SIZE RESTRICTIONS TO INSURE MODELING ACCURACY ARE:

   MAXIMUM BEAM SIZE---  3000.0

   MINIMUM BEAM SIZE--   150.0

   MAXIMUM COLUMN SIZE---  2000.0

   MINIMUM COLUMN SIZE---   150.0


**************************************** PHASE 1 ****************************************

INITIAL DESIGN COST---   1587.67

AFTER  8 ITERATIONS THE STATIONARY POINT IS ESTIMATED TO BE AT:
    710.94    696.87   2853.37   1246.53

NUMBER OF COMPLETE ANALYSES =   8


**************************************** PHASE 2 ****************************************

CONSTRAINT NUMBER  43 IS VIOLATED

SATISFACTION OF PREVIOUSLY VIOLATED CONSTRAINT  43 YIELDS THE DESIGN VECTOR:
    710.94    696.87   2853.37   1444.84

NUMBER OF COMPLETE ANALYSES =  10

AN ADJUSTMENT STEP FROM THE PRECEDING DESIGN VECTOR YIELDS:
    705.12    687.44   2858.67   1443.01

NUMBER OF COMPLETE ANALYSES =  12


**************************************** PHASE 3 ****************************************

PRESENT DESIGN COST---   1555.14
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PREVIOUS DESIGN VECTOR IS OPTIMAL IN APPROXIMATE FEASIBLE DOMAIN - BETTER APPROXIMATION BEING INITIATED

PRESENT DESIGN COST---    1555.14


GRADIENT PROJECTION ITERATION YIELDS THE DESIGN VECTOR:
    705.12     687.90    2858.50    1439.24

NUMBER OF COMPLETE ANALYSES =  14

PRESENT DESIGN COST---    1554.97


**************************************** PHASE 4 ****************************************


*********************** THE PRECEDING DESIGN VECTOR IS OPTIMAL ***********************


THE COMPLETE DESIGN VECTOR AT THE OPTIMAL POINT IS:

MEMBER MOMENTS OF INERTIA (TOP DOWN)---     705.12     687.90    2858.50    1439.24
CONNECTION STIFFNESSES (TOP DOWN)---    9.00E+09    9.00E+09

MEMBER PLASTIC SECTION MODULI (TOP DOWN)---     91.25    108.65    247.54    220.12

ACTIVE CONSTRAINTS AT THE OPTIMAL ARE:    43


STATIC LOAD CONSTRAINT VALUES:

  G(  1)=  -2937.34    G(  5)=  -6831.62    G(  9)=  -1380.49    G( 13)=  -4062.93    G( 17)=      -1.82
  G(  2)=  -1004.62    G(  6)=  -3862.12    G( 10)=  -3313.20    G( 14)=  -5446.22    G( 18)=      -1.91
  G(  3)=  -1137.34    G(  7)=  -3831.62    G( 11)=  -1553.74    G( 15)=  -4408.75    G( 19)=    -363.63
  G(  4)=  -2804.62    G(  8)=  -6862.12    G( 12)=  -3139.94    G( 16)=  -5100.40    G( 20)=    -684.15


MODERATE EARTHQUAKE CONSTRAINT VALUES:

  G( 21)=  -1806.68    G( 25)=  -1312.55    G( 29)=  -3876.94
  G( 22)=   -780.08    G( 26)=  -3245.26    G( 30)=  -5260.24
  G( 23)=  -5156.28    G( 27)=  -1349.06    G( 31)=  -3491.00
  G( 24)=  -2797.82    G( 28)=  -2935.26    G( 32)=  -4182.65
```

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STRONG EARTHQUAKE CONSTRAINT VALUES:

G(33)= -17838.55	G(37)= -211.92	G(41)= -1569.12
G(34)= -16010.31	G(38)= -2144.53	G(42)= -2952.42
G(35)= -46349.51	G(39)= -27.80	G(43)= -2.50
G(36)= -43380.02	G(40)= -1614.00	G(44)= -694.15

************************************ CONGRATULATIONS, YOU≠VE MADE IT, NOW PAY THE COMPUTER BILL ************************************

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