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DAMAGEABILITY IN EXISTING BUILDINGS

by

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1. INTRODUCTION

The Earthquake Hazards Reduction Act passed by Congress in 1977 and the Administration Plan prepared by the Office of Science and Technology [11] call for a broad program of investigation of seismic hazard. Investment in hazard abatement where only seismic hazard has been considered may, however, be unwise and futile. An overall assessment of hazard, nonseismic as well as seismic, is necessary. Such an assessment could lead to changes in priority ratings and influence decisions to abate hazard. A number of procedures by which hazard can be evaluated are needed in order to implement an investigation of overall hazard. These include procedures by which the effect of a specific hazard on a particular structure can be assessed, as well as methods by which effects of various hazards on categories of existing engineered and nonengineered structures can be evaluated.

The relative degree of hazard that buildings in a large inventory of structures represent must be identified in order to rationalize the assignment of priorities for detailed evaluation and abatement of hazard in individual buildings. A method by which relative hazard can be evaluated should meet the following requirements:

1. Simplicity: The method must be relatively simple to use; results must become available quickly.
2. Reliability: The method must account for all primary sources of hazard and must be verified against observations of damage and behavior in recent exposures of structures to different hazards.
3. Flexibility: The method must be capable of being modified easily in order to account for changes in criteria, local conditions, and improvements in modeling techniques.
4. Compatibility: The method must be compatible with the modeling of all natural hazards, e.g. earthquakes, fire, tornadoes and high winds, flood waves, storm waves, and extremes of snow and rain.

A rating system for hazard assessment has been developed for the General Services Administration [22]. In the computer-based rating system, descriptors of exposure and of building characteristics are combined with specially developed weighting factors; a vulnerability index for each hazard to be considered is calculated from these combined factors. These indices are then combined to yield an overall hazard rating on a scale of from 0 to 9. The primary advantage of this system is that it meets the requirements of simplicity, flexibility, and compatibility.

After hazardous buildings have been identified by a method such as that described above, a more refined assessment of hazard must be carried out. This can best be accomplished by a method that evaluates damageability. Lessons from the performance of buildings during recent earthquakes and other natural disasters emphasize the need for more reliable methods of evaluating damageability.

In this report a method is described by which the earthquake damageability of existing structures can be assessed. Damageability indices, used as measures of potential damage, are determined for a series of seismic events of increasing intensity. In this manner, the performance of a building exposed to a prescribed history of seismic events can be evaluated. While the determination of the damageability of an individual structure is addressed in this report, the methodology can be used to determine the damageability of classes of structures, that is, structures grouped by age and type.

1.1 Definitions

Damageability is defined as the level of damage that will occur in a building exposed to a single natural hazard or a series of such events. Damage is defined as a loss of serviceability, safety, or utility relative to some basic undamaged condition. In this sense, damage is any physical condition that requires either repair or modification to restore utility. In extreme cases, when a building cannot be repaired, demolition may be required. The term damage is frequently used in lieu of the term damageability where it is understood that future or potential damage is under consideration.

Three damageability indices are defined in this study, all of which are primarily related to damage that can be associated with structural behavior. Thus, cosmetic, functional, and socioeconomic damage are not considered. The indices are: local damageability - r_i ; global damageability - R_g ; and cumulative damageability - r_{ic} or R_{gc} , local or global cumulative damageability, respectively.

<u>Local or element damageability</u> is a measure of damage to a single building element, part of an element, or a group of similar elements. The index r_i is a function of a demand parameter d_i , a capacity parameter c_i^o at which damage is initiated, and a capacity parameter c_i^u at which damage is total. Thus, the index r_i is a function normalized to vary between 0 (no damage) and 1.0 (total damage), i.e., $0 \leq r_i = f(d_i, c_i^o, c_i^u) \leq 1.0$.

<u>Global damageability</u> is a measure of damage to a structure as a whole, or, in special cases, to a large portion of a structure comprised of many elements. Global damageability must reflect not only the mean value of local damageabilities, but also the relative importance of individual elements within the global system. Each element is assigned an importance factor p_i , and the index R_g is a function normalized to vary between 0 (no damage) and 1.0 (total damage), i.e. $0 \leq R_g = f(p_i, r_i) \leq 1.0$

<u>Cumulative damageability</u> is a measure of cumulative damage to a structure as the result of previous loading or hazardous events. Thus, given damageability r_i^o based on previous damage, the cumulative index r_{ic} becomes

$$0 \leq r_{ic} = f(r_i^o, d_i, c_i^o, c_i^u) \leq 1.0$$

where the function is normalized to vary between 0 and 1.0.

Similarly, the cumulative global index R_{gc} is defined by:

$$0 \leq R_{gc} = f(p_{ic}, r_{ic}) \leq 1.0$$

where p_{ic} is an importance factor associated with cumulative damage.

2. SCOPE

Although the damageability of a structure involves the interrelationship of probabilistic events and resulting probabilistic response, the proposed methodology is deterministic except for the prediction of future ground motion, e.g. response spectra. Estimates of most probable characteristics of ground motion are therefore used where appropriate. For a summary of methods by which damageability may be evaluated using probabilistic approaches, the reader is referred to Yao [24]. Czarnecki et al. also summarize methods, both deterministic and probabilistic, used by URS/John A. Blume & Associates [4, 6, 12, 21].

In the research reported here, the damageability of a structure is determined from the damageability of elements within the structure. Bertero and Bresler [1] introduced the concepts of local and global damageability indices, but did not develop a method of evaluation. In this report, a procedure is set forth in which structural response is assessed, damageability is evaluated, and the demand on a structure for a specified earthquake force is calculated.

The damageability of both structural and nonstructural members is considered. The terms primary structural and secondary structural are used to distinguish between members whose stiffness is included in the structural response model and those whose stiffness is not. Mechanical, electrical, and architectural elements that do not contribute to structural response are termed nonstructural members; the damageability of such nonstructural elements is assessed, when deemed necessary, using the model for structural response.

The evaluation is based on a sequence of analyses, each more refined than the last. This sequence of analyses was termed screening by Okada and Bresler [16]. Structural damage models of varying degrees of sophistication can be used. The effects of previous loading, e.g. loadings from fires and earlier earthquakes, can be considered [7].

Static structural analyses for specified levels of loading are performed during the design process; the prevention of excessive deformation, yielding, and/or collapse is emphasized. Damageability should, however, be evaluated for a more complete spectrum of possible loadings. A large number of time-history response analyses could be carried out in order to determine damageability for a wide range of earthquake forces, although the expense would be considerable. A more practical approach is proposed in this report. Quasi-static analyses are performed in which damageability is evaluated for a continuously increasing load. Approximate methods by which loading is related to future earthquakes of varying probability of occurrence are proposed. The type of damageability-response-demand relationship that is sought is illustrated in Figure 2.1. Although the methodology is intended for a first or early screening for which a quasi-static response analysis is performed, some of the procedures are applicable to more sophisticated screenings for which time-history response analyses must be performed.

The evaluation is divided into three steps. A structural analysis in which current modeling techniques are employed is first conducted. The quasi-static response analysis proposed here is a variation on elasto-plastic or piecewise-linear analyses. The approximation of earthquake-induced inertial forces is such that the lateral response of the structure can be expressed in terms of a single generalized coordinate and a single generalized magnitude of force. The area under the curve force-displacement relationship thus obtained represents the potential energy of the structure. The force-displacement relationship for an equivalent single-degree-of-freedom system is obtained when the quantities are normalized.

The damageability of a structure as a function of displacement is determined. Forces, displacements, and accelerations throughout the structure can be calculated readily. The local damageability indices for elements throughout the structure are determined by combining capacity and demand parameters using simple piecewise-linear damageability relationships. The local indices are then combined with importance factors to form a global damageability index that represents the damageability of the structure as a whole. The index is normalized such that a value of one indicates irreparable damage and a value of zero indicates that no damage has occurred.

The damageability of a structure is related to potential earthquake demand by inelastic response spectra. The force-displacement relationship for the equivalent single-degree-of-freedom system is compared with force-displacement curves obtained using the response spectrum. The level of response for the equivalent system that corresponds to the particular spectrum is estimated. The global damageability index for the estimated level of response is determined from the damageability relationship described above.

For a later quasi-static screening, a modification of the procedure described above is proposed. The modification reflects the interaction of earthquake demand and structural capacity; that is, as the stiffness of a structure decreases with increasing load, the redistribution of inertial force is considered.

3. DAMAGEABILITY INDICES

3.1 Element or Local Damageability

It is reasonable to describe potential damage to a structure, i.e. its damageability, in terms of the damageability of the elements of which the structure is comprised. An element can be a single member, a group of similar members, or part of a member, depending on the sophistication of the damage model and engineering judgment. In this discussion, the damageability of an element is termed local damageability. Primary structural members are considered in both the structural and damage models. Members that do not contribute significantly to structural response are termed secondary structural members and may be considered in the damage model if damage to them is deemed important. The term secondary structural element encompasses those permanent and semipermanent elements that are sometimes termed nonstructural. The term nonstructural is reserved for mechanical, electrical, and architectural elements that do not participate in structural response. Any nonstructural element for which an assessment of damageability is desired can be included in the structural model.

An index is used to measure local damageability. For measurements of local damageability to be meaningful physically, certain factors should be taken into account. First, damage to some elements, such as ductile beams, is progressive, while damage to other elements, such as unreinforced masonry partitions, is sudden and complete. A damageability index should accommodate both types of damage. An index should also consider the current condition of the structure as well as its condition at some point in the future if deterioration due to normal service conditions or to exposure to hazard is likely.

Local damageability indices will be determined from structural response parameters since it is assumed that structural response (forces, displacements, velocities, accelerations) can be related to the response of secondary and nonstructural elements. Damage is in fact a complex function of these and other parameters.

A simplified procedure is proposed here whereby damageability can be measured. For each element, one or more response parameters are combined to form a damage demand parameter. This demand parameter can be as simple as a single nodal displacement or as complex as a combination of several response parameters, e.g. forces, moments, displacements. The damage capacities of an element in terms of the demand parameter are determined or estimated. A piecewise-linear damageability model is suggested; this model requires that a value of the demand parameter at which damage is initiated and a value at which damage is total and irreparable be specified. The piecewise-linear model can then be expressed as follows:

$$r_i = \frac{d_i - c_i^o}{c_i^u - c_i^o} \tag{3.1}$$

where r_i is limited by

$$0 \leq r_i \leq 1 \tag{3.2}$$

and where

r_i - local damageability index for the i^{th} element;

d_i - demand parameter from a combination of response parameters;

c_i^o - capacity at which damage is initiated (this value may be calculated or may be a best guess);

c_i^u - capacity at which damage is complete, ultimate, or irreparable (this value may also be calculated or may be a best guess).

While the value of c_i^u is generally greater than that of c_i^o, the values may be identical for a brittle element. As the value of the demand parameter increases from zero to c_i, the value of the damageability index remains at zero since it cannot be negative until the value at which damage is initiated is reached. The index then increases linearly until ultimate capacity is reached. The index then is set equal to one at which value it remains despite further increases in the demand parameter. Physically, a value of zero for the index suggests that no damage will occur and a value of one indicates that complete, irreparable damage will occur.

Where appropriate, more than one criterion can be established for the damageability of a single element. The worst or weighted sum of the indices from each criterion can be used to determine the local damageability index. Even when several indices are combined for a single element, the limits specified in equation 3.2 are used to determine the resulting local index.

Equation 3.1 can be modified to consider damage that has resulted from previous loadings. If current damageability is not a direct function of previous damage, the index can be determined as follows:

$$r_i = r_i^o + \left(\frac{d_i - c_i^o}{c_i^u - c_i^o}\right) (1 - r_i^o) \qquad (3.3)$$

where the current condition of a structure has been considered in determining d_i, c_i^o, and c_i^u, and r_i^o is the prior local damageability index (at the time of the assessment). The limits expressed in equation 3.2 pertain also to the above equation.

3.1.1 Local Damageability of an Infilled Wall

A section of an infilled wall, illustrated in Figure 3.1, is assumed to be a secondary structural element; that is, the wall is assumed not to contribute to the stiffness of the structure. Although the stiffness of such a wall might well contribute to the response of a structure, this stiffness may justifiably be neglected for an early screening.

The damage to the wall is determined from available response parameters. Bertero and Bresler [1] suggest that average tangential drift be adopted as a damage parameter. The demand parameter d can then be calculated as follows:

$$d = \frac{(u_3 - u_1)}{H} + \frac{(u_6 + u_8 - u_2 - u_4)}{2L} \qquad (3.4)$$

where

- d - average tangential drift index - demand parameter;
- H - interstory height;
- L - length of wall
- u - horizontal or vertical nodal displacement as illustrated in Figure 3.1.

The values of the average tangential drift index at which damage is initiated (c^o) and at which damage is complete (ultimate damage c^u) must be estimated from experimental and/or analytical data.

The piecewise-linear damageability profile for the infilled wall is illustrated in Figure 3.2. Damage may occur in steps, as suggested by the dotted lines in that figure. The determination of the demand values at which these steps will occur requires that experimental data be available. Since there will generally be considerable scatter in such data, the use of a piecewise-linear relationship would seem to be justified. Damage to walls made of certain materials (such as unreinforced masonry) will be sudden and complete; the sloped portion of the indicial relationship in Figure 3.2 is vertical for such walls.

3.1.2 Local Damageability of a Column Element

A column element can fail due to excessive shear or can be damaged due to the interaction of axial and flexural loads. Although shear and flexural-axial damageabilities are related, they are sufficiently independent to warrant separate damage criteria. Therefore, both shear and flexural-axial indices must be determined for column elements, with the highest index taken as the local damageability index.

The internal load on a column element is often determined from a model in which inelastic behavior is limited to the ends of the element. Severe damage is thus limited to these discrete locations. A local damageability index is determined for each potential hinge location; each index therefore represents one-half of the structural element.

Flexural-axial interaction can be modeled by the column interaction diagram shown in Figure 3.3. If the combined axial load and moment remain within the interaction curve defined by code limit values with appropriate capacity reduction factors, damage is assumed to be negligible. When the combination first reaches the curve, damage is assumed to have been initiated. Ultimate damage corresponds to an envelope defined by calculated ultimate values where a capacity reduction factor of 1.0 has been assumed.

Although the interaction curve is illustrated in terms of axial force P and bending moment M, a relationship between P and curvature $\dot{\theta}$ can be used as an alterantive to determine the extent of damage. For structural modeling in which the curvature after initial hinging can be determined, the damageability index is assumed to be a linear function of the curvature relative to the curvature at initial hinging. If the column is modeled by an idealized elasto-plastic element, the curvature at the hinge is undefined; the relative angle across the hinge is used as the demand parameter [13]. The damageability index for flexural-axial damage is illustrated in Figure 3.4, where the dotted line again suggests that damage occurs in a more irregular manner than that indicated by the solid curve.

The failure of a column in shear is assumed to be sudden and complete at a specified value of shear force (Figure 3.5). If a shear failure occurs, the value of the local damageability is set equal to one regardless of the interaction of axial and flexural forces.

3.2 Global Damageability Index

The local damageability indices are combined to form a global index, a measure of the damageability of the structure as a whole. When determined in conjunction with a quasi-static structural analysis, the local indices can be evaluated as continuous functions of the response parameters; a global index that varies with the magnitude of structural response and loading is therefore possible.

The global damageability index should reflect the differing importance of elements within the global system [1]. There should also be upper and lower limits to the global index with readily interpreted physical significance. A procedure by which a global damageability index can be determined is described below.

Global boundaries are determined first. Generally, the boundaries will encompass a structure as a whole, but for special problems the boundaries may be for a critical part of a structure only, e.g. the first floor; global boundaries may also be defined for several structures that are attached at some point. Each element within the global boundaries, including primary and secondary structural members, is assigned an importance factor p_i. The importance factor reflects life hazard, replacement cost, or other considerations important for a particular structure. Elements that if damaged totally might induce catastrophic damage, i.e. structural collapse, would be assigned high importance factors and would also be starred for special consideration as will be described later.

The global damageability index is defined as follows:

$$R = \frac{\sum_n (p_i r_i)}{\sum_n p_i} \qquad (3.5)$$

where

R - global damageability index;

p_i - importance factor for i^{th} element;

r_i - local damageability index for i^{th} element;

$\sum_n$ - summation over all elements within the global boundaries.

The value of R thus varies between zero and one, where a value of zero indicates no damage and a value of one indicates total damage to all elements. Critical elements are checked to determine if a combination of damage to such elements could precipitate collapse or render a structure irreparable. If so, the global damageability index is set equal to one.

3.3 Structural Response Requirements for Damageability Assessment

Response parameters must be available for each level of response for which an assessment of damageability is desired. These parameters include the displacements and accelerations throughout a structure and the loads on primary structural members. In a quasi-static analysis, a single response parameter, e.g. base shear, can characterize the level of response and magnitude of lateral inertial force. When the response parameters used to assess damageability are represented as functions of a single characteristic parameter, a continuous representation of global damageability as a function of the characteristic parameter is possible.

For the damageability indices to be meaningful physically, the relationship between the indices and earthquake demand must be established. An available analytical tool by which earthquake demand can be estimated is a response spectrum. However, a single-degree-of-freedom representation, or a combination of such representations, of a structural system must be adopted in order to use response spectra for this purpose. In the sections that follow, we will develop an approximate method by which structural response can be measured in terms of the response at a single coordinate.

4. STRUCTURAL MODELING

4.1 Modeling Considerations

The structural model of a building or other existing structure is often constructed from elements, where for the purpose of this discussion the term element is taken to mean a single structural member, a group of structural members, or a portion of a member, depending on the degree of sophistication or refinement of the model. Generally, the smaller the section of a structure represented by an element (other factors being equal), the greater the sophistication of the model, but the more expensive and/or complex the response analysis. The degree of refinement of a structural model should be such that the model is consistent with the level of knowledge or predictability of the forcing function and accounts for factors that are expected to be important. Thus, if the forcing function can be estimated only within broad limits, as is frequently the case, a detailed structural model may not be warranted. An additional factor to be considered is the availability of appropriate computational aids, such as computer programs for the response analysis, subroutines for the analysis of individual structural elements, and/or analytical models for individual elements.

The structural model of an existing building should represent the current condition of the building or the condition of the structure at the time for which an assessment of damageability is desired. The state of the structure due to previous loadings, e.g. earthquake or fire loadings, should be reflected in the specification of material properties, effective cross sections, and/or in other modeling parameters. While average or effective properties are often used to model structural members, the effects of previous loadings should be considered when these properties are specified.

The structural properties of elements can vary or change in response to severe earthquake excitation. In the discussions that follow, piecewise-linear stiffness and inelastic behavior are considered for a monotonically increasing load, but the degradation of stiffness under cyclic loading is neglected. Two or more models may be necessary to approximate the effect of stiffness degradation in a quasi-static analysis. The consideration of brittle elements that fail suddenly can also require that two models be used for each brittle element, one in which the element is included and another in which it is considered to have been removed or its load-carrying capacity to have been greatly reduced.

4.2 Model for a First Screening

A simple model that could be used as an initial screening is illustrated in Figure 4.1. An existing structure is modeled as a shear building with only the lateral planar motion of each floor considered. The mass of the structure is lumped at each floor. The stiffness between floors is represented by a single element characterized by a piecewise-linear force-deflection relationship. Thus, this element represents all structural elements that provide shear stiffness between floors.

The utility of this model for damageability assessment will depend on several factors. The shear building approximation must adequately represent the deformation of the structure in response to earthquake excitation. Interstory drift and lateral story acceleration are the only response parameters used to assess damage; damage in individual structural elements will be difficult to ascertain. Force and deflection curves for the entire structure must be available to use the model. In general, this model will be useful for rapid hand calculations, but more detailed models should be adopted if computer programs are used.

4.3 Model for an Early Screening

An alternative model for an early screening is a planar rectangular frame model (Figure 4.2). This model is similar to those linear and nonlinear models that are presently incorporated in time-history earthquake response analyses. While such a model may not be suitable for certain structures, it can nonetheless be widely applied.

All primary structural members are represented by one or more elements. Various levels of sophistication are possible. The models illustrated in Figures 4.3 to 4.5 are discussed in detail in reference 13. For the beam model in Figure 4.3, it is assumed that inelastic response is confined to discrete locations at the ends of the free span of the beam. A bilinear relationship between moment and curvature is approximated by combining elastic and elasto-plastic components [13]. The column element yield criteria for the elasto-plastic component are represented by the interaction curves in Figure 4.5.

The capacity of computer programs in which only linear-elastic response is considered is generally greater than that of programs in which nonlinear or inelastic response is considered; the former will also be less expensive to use. A detailed elastic model should therefore be established and analyzed first. If maximum earthquake loadings do not result in predictions of element response beyond the elastic limit, a nonlinear analysis will not be necessary. If a nonlinear analysis is necessary, bilinear element models can be analyzed for the next screening. More sophisticated models are of course possible. A three-dimensional model may be necessary for certain structures, even for an initial screening, in order to account for large torsional excitation. For other structures, the response of shear and brittle elements should be considered. In the discussion that follows, the two-dimensional frame model is emphasized.

5. EARTHQUAKE STRUCTURAL DEMAND

5.1 Available Analytical Methods

Although elaborate computer programs are available by which the time-history nonlinear response of structures to a prescribed earthquake ground motion can be determined, e.g. [13], their use cannot always be justified. Due to the uncertainty associated with the characteristics of future ground motion--soil-structure interaction, structural damping, participation of secondary elements in structural response--as well as the high cost of a sophisticated nonlinear analysis, a static or quasi-static analysis is often used to predict maximum structural response to anticipated earthquake ground motion. The inertial forces created by an earthquake excitation and subsequent dynamic response are estimated and applied as static external forces that act on the structural model. When varying levels of force are considered, the analysis is quasi-static, i.e. the forces are increased slowly so as not to introduce additional dynamic effects. At levels of applied force at which element properties change, the structural model is updated and forces are further increased. Brittle elements from which stored energy is released at a single level of loading require special consideration.

Two difficulties inherent in a static or quasi-static analysis are the estimation of the distribution of force by which the distribution of inertial force during dynamic earthquake response can be estimated and the estimation of the magnitude of these forces that corresponds to an expected future earthquake.

5.1.1 Linear-Elastic Structural Response

If a structure under consideration is assumed to respond in a linear-elastic mode throughout an earthquake excitation, structural response can be evaluated by standard methods. The response of a linear-elastic structure is often represented as occurring through a finite number of natural modes of a structure. If damping assumes specific forms, the differential equations of motion are uncoupled by modal transformations so that the response in each mode of the structure can be analyzed separately. Modal analysis of structural response to earthquake excitation is discussed in references 8 and 9.

Although the time-history response of a linear-elastic structure to a specified ground motion can be determined using modal analysis, the maximum response of a structure is of primary importance. The maximum response of a single-degree-of-freedom system to a specified ground motion can be obtained from response spectra. A response spectrum is the maximum or nearly maximum, response of a single-degree-of-freedom system as a function of the natural period of the system [2, 8, 9, 15]. Response spectra for anticipated earthquakes can be predicted or estimated without the necessity to generate time histories of ground motion. Several spectra with varying probabilities of occurrence can be used. Response spectra are therefore a potentially useful tool for the assessment of damageability. For a structure that will respond in one or more uncoupled modes, a response spectrum can be used to predict the maximum contribution from each mode to a response quantity. The maximum response must, however, be combined by a probabilistic method such as the square root of the sum of the squares procedure [15] since the maxima of all modes will generally not occur simultaneously. The forces for each mode can be combined to yield a set of equivalent forces since a unique set of inertial forces corresponds to the maximum response in each mode.

Building codes specify the distribution and magnitude of lateral force for earthquake-resistant design, but the magnitude of such design loadings is generally low compared to that of severe earthquake excitations. Codes imply that a structure will respond elastically to design loadings, but in fact rely on inelastic response to absorb the additional demands that will be placed on structural systems by severe earthquake loading.

The sum of the lateral design forces equals the design shear force at the base of the structure. This base shear can be divided by the mass of the structure to obtain an average lateral acceleration that can be represented in g's. A distribution of forces as specified in a code can be used to assess damageability, but several g-loadings that realistically correspond to earthquakes with varying probabilities of occurrence should be determined as well. While maximum ground acceleration during a time-history response is not equivalent to these g-loadings, for many characterizations of earthquake loading and structural type, empirical or probabilistic relationships between these quantities can be established.

5.1.2 Inelastic Structural Response

If a structure is expected to respond inelastically to dynamic loading, a structural model in which inelastic response is considered should be used in an assessment of damageability. The determination of equivalent earthquake forces for such structures is complex. The differential equations of motion above the elastic limit are not uncoupled by elastic modal transformations.

If it is assumed that the distribution of equivalent lateral force does not change for inelastic behavior or that it changes in a known manner, a quasi-static analysis can be conducted by slowly increasing the load until an element stiffness is estimated to change. The stiffness of the structure is modified and the loading again increased until the next change in stiffness is predicted to occur and the model is again updated. The procedure can be repeated until the base shear, or g level, correponds to a predicted level of probability of occurrence or until structural collapse is predicted. A computer program in which such a response analysis is incorporated is ULARC [19].

If the structural system is such that it can be represented by a single degree of freedom, various linear and inelastic response spectra are available by which maximum response can be predicted. A linear response spectrum can be utilized by matching the stored energy of the inelastic system, as represented by the area under the force-deflection curve for the structure, to that of an elastic system of the same frequency. This procedure is believed to yield acceptable results over certain frequency ranges. For elasto-plastic systems, however, a more general response spectrum has been developed by Newmark [15]. The spectrum comprises three portions, one each for low-, intermediate-, and high-frequency systems. For each portion it has been assumed that either the maximum displacement, energy, or acceleration is identical to that from a linear response spectrum. A different spectrum is obtained for each ductility factor considered.

Response spectra for a trilinear system with characteristics that correspond to specific earthquake input have also been developed [14, 20]. Again, a single-degree-of-freedom model or analogy must be used.

A reserve energy technique by which earthquake loads in the inelastic range can be evaluated has been developed by Blume [3, 5]. Although the method is approximate for multistory buildings, the basic mechanisms by which structures resist severe earthquake ground motion are accounted for.

5.2 Representation of Inertial Force by Lateral Loads

For structures that can be expected to respond inelastically to severe dynamic loading, it is unclear by what distribution and magnitude of lateral force a predicted earthquake excitation can best be represented. An equivalent single-degree-of-freedom system is required if available response spectra for inelastic systems are to be used. For many low- to medium-rise structures the response in the fundamental mode largely determines the overall response of a structure to a typical earthquake excitation. In what follows, fundamental modal response will be used in deriving a single-degree-of-freedom analogy. Inelastic behavior will be accounted for by considering the changes in the fundamental mode during inelastic response. In this way, available inelastic response spectra can be used to evaluate response.

6. QUASI-STATIC RESPONSE ANALYSIS

6.1 Introduction

A response analysis is described here in which inelastic element behavior is considered; the procedure is such that a single-degree-of-freedom analogy can be drawn which is suitable for use in certain methods of predicting earthquake demand, e.g. response spectra. The response of a structure in one plane only is considered. The structure is modeled by primary structural elements such as those defined in section 4. The inelastic behavior of elements is assumed to occur at discrete locations. The representation of the stiffness of each element is bilinear; the resulting global stiffness of the structure is thus piecewise linear. All primary structural elements are assumed to be ductile for the discussions in sections 6.2 through 6.4. The inertial forces that result from earthquake excitation are assumed to occur at a finite number of locations, usually at each floor of a structure. The effects of gravity, including those of geometric stiffness, are accounted for in the analysis.

The lateral motion of a structure is assumed to occur in a single mode or pseudo-mode of the structure in order to derive the single-degree-of-freedom analogy. Distributions of lateral load by which only the fundamental mode is deflected are increased quasi-statically. Inelastic response is reflected through changes in the frequency of a mode. A refinement that reflects changes in the distribution of lateral force due to inelastic behavior is provided in higher level screenings.

The motion of many structures, e.g. that of most high-rise buildings, cannot be adequately represented by motion in the fundamental mode alone. The procedure described below can be considered an initial screening for such structures; this initial screening would be followed by a more elaborate time-history or other analysis. The procedure is intended primarily for low- to medium-rise structures for which response is dominated by the fundamental mode.

6.2 Procedure for Initially Elastic Structures

The global stiffness matrix is assembled from element stiffnesses. The effects of previous loadings, such as those from fire and earthquakes, are considered in the selection of material properties. A mass matrix, specified only in terms of geometric coordiates at which lateral earthquake forces are applied, is assembled.

The effect of gravity loads is determined. The combined gravity forces and structural stiffness terms should represent the condition, internal loads, and displcements of the existing structure as closely as possible. In addition to previous transient loads, which can affect material properties, the order in which the structure and any additions to the structure were constructed can significantly influence the internal loads on the structure due to gravity.

Geometric stiffness terms can be added to the stiffness matrix to approximate the effect that axial loads have on large lateral displacements, i.e. so-called P-Δ effects. Although geometric stiffness is a function of axial load, and axial load is a function of lateral displacement, sufficient accuracy is often achieved when only the axial load caused by gravity forces is considered. If results obtained when this approximation is assumed are not sufficiently accurate, the geometric stiffness can be updated at intervals of lateral displacement during the quasi-static analysis.

The global stiffness for a structure is reduced to the lateral coordinates at which mass has been assigned. The reduction transformations are retained so that once the reduced set of lateral displacements and accelerations has been determined, global response can be evaluated.

The modes and frequencies are determined from the equation

$$[M] \{\ddot{x}\} + [K] \{x\} = \{0\} \tag{6.1}$$

where $\{x\}$ is a vector of lateral geometric displacements, $[M]$ is a mass matrix, and $[K]$ is a reduced global stiffness matrix. The resulting orthogonalized transformation is

$$\{x\} = [\phi] \{q\} \tag{6.2}$$

where $\{q\}$ is a vector of modal displacements and $[\phi]$ is a matrix of energy-normalized modes such that

$$[\phi]^T [M] [\phi] = [I] \tag{6.3}$$

where [I] is an identity matrix. The resulting modal stiffness matrix is

$$[\phi]^T \; [K] \; [\phi] \;=\; \lceil \omega^2 \rfloor \tag{6.4}$$

where $\lceil \;\; \rfloor$ is a diagonal matrix and the ω^2 are the squares of the circular frequencies of the modes.

The distribution of initial lateral force used to approximate the earthquake-induced inertial forces is chosen so as to cause response only in the first mode since lateral motion is assumed to occur only in that mode:

$$\{F\} \;=\; [M] \; \{\phi_1\} \; f \tag{6.5}$$

where $\{F\}$ is a vector of lateral forces applied for the first increment of response, $\{\phi_1\}$ is the fundamental mode shape, and f is a scalar multiplier for the first increment of response. Initially, f is set equal to one; later, the value is set equal to the elastic limit.

The generalized forces are

$$\{Q\} \;=\; [\phi]^T \; [M] \; \{\phi_1\} \; f \tag{6.6}$$

or, from the orthogonality relations,

$$\{Q\} \;=\; \begin{Bmatrix} f \\ 0 \\ 0 \\ \cdot \\ \cdot \\ \cdot \\ 0 \end{Bmatrix} \tag{6.7}$$

For quasi-static conditions, the modal displacements are determined from

$$[\;\; \omega^2 \;\;] \; \{q\} = \{Q\} = \begin{Bmatrix} f \\ 0 \\ 0 \\ \cdot \\ \cdot \\ \cdot \\ 0 \end{Bmatrix} \tag{6.8}$$

The modal displacement in the first mode therefore is

$$q_1 \;=\; \frac{f}{\omega_1^2} \tag{6.9}$$

and the modal displacement in all higher modes is zero. The application of lateral force is represented in Figure 6.1.

The lateral displacements at the reduced set of global coordinates can be determined from the first mode shape:

$$\{x\} = \{\phi_1\} q_1 \tag{6.10}$$

The applied lateral forces are an approximation of the inertial earthquake forces; the accelerations necessary to induce such forces can therefore be determined as follows:

$$[M] \{\ddot{x}\} = [M] \{\phi_1\} f \tag{6.11}$$

or

$$\{\ddot{x}\} = \{\phi\} f = \{x\} \omega_1^2 \tag{6.12}$$

The reduction transformations are used to determine the displacements and accelerations at all global coordinates. The internal load on each structural element is determined by global-to-element coordinate transformation. The increments of internal load and displacement are added to the values due to gravity load.

The structural elements are examined to determine the lowest magnitude of force f_1 that will cause the elastic limit of the element to be exceeded. For certain elements, such as beam-column elements, interaction curves are required in order to determine the magnitude of this force. This lowest value of f_1 is used to determine displacements, accelerations, and internal loads. These response quantities represent the limit of linear-elastic quasi-static response.

The energy input to a structure by lateral forces is the integral of forces over displacements, or

$$E = \frac{1}{2} \{F\}^T \{x\} \tag{6.13}$$

or

$$E = \frac{1}{2} f \{\phi_1\}^T [M] \{\phi_1\} q_1 \tag{6.14}$$

or

$$E = \frac{1}{2} f q_1 \tag{6.15}$$

Thus, stored potential energy can be represented as the area under a force-displacement curve as shown in Figure 6.2. The equation of motion in terms of the modal coordinate q_1 differs from that of a simple single-degree-of-freedom oscillator by a modal participation factor $(\{\phi\}^T\{M\})/(\{\phi\}^T[M]\{\phi\})$ where the denominator has been normalized to unity and M is a vector of lumped masses of the structure. Force and displacement are therefore modified as follows for the single-degree-of-freedom analogy:

$$\bar{f} = \frac{f_1}{\{\phi\}^T\{M\}} \tag{6.16}$$

and

$$\bar{q} = \frac{q_1}{\{\phi_1\}\{M\}} \tag{6.17}$$

where $\bar{f}$ and $\bar{q}$ are the equivalent force and displacement of a single-degree-of-freedom oscillator. The potential energy of the equivalent system is:

$$E = \frac{1}{2}\bar{f}\,\bar{q} \tag{6.18}$$

as represented in Figure 6.2 by the second set of scales.

6.3 Increment of Nonlinear Response - Constant Shape Force Distribution

If the shape of the distribution of lateral force is assumed to be identical for elastic and inelastic responses, a global stiffness matrix based on a displacement greater than that of the previous level can be assembled. The stiffness values consider the effects of previous loading increments as reflected in element stiffnesses modified to account for inelastic behavior. Geometric stiffnesses can be included if P-Δ effects are expected to be significant. The modified global stiffness matrix is reduced to the lateral coordinates to which mass has been assigned. The transformations are retained.

An equivalent modal stiffness matrix is formed:

$$[K_i^*] = [\phi]^T\ [K_i]\ [\phi] \tag{6.19}$$

where the subscript i indicates the i^{th} increment, $[K_i^*]$ is the equivalent modal stiffness matrix, and $[K_i]$ is the reduced discrete stiffness matrix for the i^{th} increment. The equivalent modal stiffness matrix is segmented as follows:

$$[K_i^*] = \left[\begin{array}{c|c} K_{11}^* & K_{12}^* \\ \hline K_{21}^* & K_{22}^* \end{array}\right]_i \tag{6.20}$$

where K_{11}^* and K_{22}^* are square and K_{11}^* is of order one.

The shape of the distribution of lateral forces is assumed to be the same as that for the elastic response increment; that is,

$$\{\Delta F_i\} = [M]\ \{\phi\}\ \Delta f_i \tag{6.21}$$

where Δ indicates the incremental quantity. Subscripts that indicate the first mode, such as for the vector $\{\phi\}$, are implied in the following discussion where ambiguity is unlikely.

The generalized forces for the elastic modes are given by

$$\{\Delta Q_i\} = \begin{Bmatrix} f_i \\ 0 \\ 0 \\ \cdot \\ \cdot \\ \cdot \\ 0 \end{Bmatrix} \tag{6.22}$$

Since generalized forces are applied only in the first mode, the equivalent modal stiffness matrix can be reduced to the first mode as follows:

$$\omega_i^2 = K_{11}^* - \{K_{21}^*\}^T\ [K_{22}^*]^{-1}\ \{K_{21}^*\} \tag{6.23}$$

where ω_i^2 is the effective stiffness collapsed to the first mode. The effect of any change in stiffness due to inelastic element response is thus represented by a change in the effective stiffness, i.e. the frequency, in that mode. A displacement in the first mode implies a corresponding displacement in higher modes.

The incremental displacement in the elastic first mode due to lateral forces is

$$\Delta q_{1_i} = \frac{\Delta f_i}{\omega_i^2} \tag{6.24}$$

where the subscript i refers to the i^{th} increment and not the i^{th} mode.

The displacement in the higher elastic modes is determined as follows:

$$\begin{Bmatrix} \Delta q_2 \\ \Delta q_3 \\ \cdot \\ \cdot \\ \cdot \\ \Delta q_n \end{Bmatrix} = [K^*_{22}]^{-1} \{K^*_{21}\} \Delta q_{1_i} \tag{6.25}$$

The i^{th} increment of lateral force and displacement is illustrated in Figure 6.3.

The incremental displacements and accelerations at the lateral coordinates are determined from the following modal transformations:

$$\{x_i\} = [\phi] \{\Delta q\}_i \tag{6.26}$$

and

$$\{\ddot{x}_i\} = \{\phi_1\} \Delta f_i \tag{6.27}$$

Since the shape of the lateral force distribution is assumed to remain constant, the geometric displacements will be linear combinations of all elastic modes, but the accelerations will be a scalar times the first mode only.

The reduction transformations are used to determine displacements and accelerations for all global coordinates. Incremental internal loads on all structural members are determined. The incremental response to a unit force f_i is added to the previous response. The magnitude of lateral force that causes a change in stiffness of the most critical structural element is determined. The incremental and total displacements, accelerations, and internal member loads are proportioned to match this force value.

Since a generalized force is applied in the first mode only, energy is not stored in the higher modes. The energy for the i^{th} increment is

$$E_i = \sum_{j=i}^{i} E_{ji} \tag{6.28}$$

where, for $j \neq i$,

$$E_{ji} = \Delta f_j \, \Delta q_{1_i}$$

and

$$E_{ii} = \frac{1}{2} \Delta f_i \, \Delta q_{1_i}$$

The force-displacement and energy relationships are represented in Figure 6.4.

The modal participation factors for the increments of inelastic response are identical to those for the linear system. The displacements and force in an equivalent single-degree-of-freedom oscillator are therefore:

$$\Delta\bar{f}_i = \frac{\Delta f_i}{\{\phi_1\}^T\{M\}} \tag{6.30}$$

and

$$\Delta\bar{q}_i = \frac{\Delta q_i}{\{\phi_1\}^T\{M\}} \tag{6.31}$$

Increments of response are determined and added to previous response quantities until a prescribed level of loading has been reached or until the structure is predicted to collapse. The force-displacement relationship can be adjusted to represent a single-degree-of-freedom oscillator as represented by the second set of scales in Figure 6.4. Since the lateral forces are approximate, the equivalent oscillator can be used to predict maximum response from available response spectra or other response predictors.

6.4 Increment of Nonlinear Response - Modified Force Distribution

In the quasi-static analysis described in the previous section, lateral earthquake forces were approximated by a constant shape distribution with an increasing magnitude. In fact, inelastic element response causes the distribution of lateral force to vary. The distribution frequently changes so as to magnify the effect of local inelasticity. For example, if a so-called weak story forms in a shear building during an earthquake, the upper floors will frequently displace with a large rigid-body contribution and little relative deformation. The distribution of inertial force will therefore change so that the critical floor will absorb an even higher percentage of energy than if the distribution had remained the same.

The quasi-static response analysis is here modified so that the redistribution of lateral force caused by inelastic element behavior can be approximated. The distribution is based on a fundamental mode shape derived from the updated incremental stiffness matrix. The shape of the incremental accelerations and displacements is thus constrained to be the same, and the incremental force distribution amplifies the effect of the reduced stiffness.

The procedure is similar to that for a constant force distribution. The equivalent modal stiffness matrix is determined; that is,

$$[K_i^*] = [\phi]^T [K_i^*] [\phi] \tag{6.32}$$

where, as before, $[\phi]$ is a matrix of elastic mode shapes and i refers to the i^{th} increment. Pseudo-modes are found from an eigenvector/function solution of the following equation:

$$[I] \{\Delta\ddot{q}\}_i + [K_i^*] \{\Delta q\}_i = \{0\} \tag{6.33}$$

where $\{\Delta q\}$ is the vector of modal displacements in the elastic modes. The resulting modal transformation is

$$\{\Delta q\} = [\Phi] \{\Delta\xi\} \tag{6.34}$$

where $[\Phi]$ is a matrix of pseudo-mode shapes and $\{\Delta\xi\}$ is a vector of pseudo-mode displacements. The following orthogonalization and normalization are assumed:

$$[\Phi]^T [I] [\Phi] = [I] \tag{6.35}$$

and

$$[\Phi]^T [K^*] [\Phi] = \lceil \Omega^2 \rfloor \tag{6.36}$$

where the Ω are the circular frequencies of the reorthogonalized system. The elastic modes and pseudo-modes are combined to form a transformation from geometric coordinates to pseudo-modal coordinates as follows:

$$\{\Delta x\} = [\Phi] \{\Delta q\}_i \tag{6.37}$$

or

$$\{\Delta x\} = [\phi] [\Phi] \{\Delta\xi\} \tag{6.38}$$

or, letting

$$[\chi] = [\phi] [\Phi] \tag{6.39}$$

$$\{\Delta x\} = [\chi]\ \{\Delta\xi\} \tag{6.40}$$

The matrix $[\chi]$ refers to the combined modes.

The lateral forces by which inertial earthquake forces are approximated are assumed to cause displacement only in the first combined or pseudo-mode; that is,

$$\{\Delta F_i\} = [M]\ \{\chi_1\}_i\ \Delta f_i$$

where the subscript 1 will be dropped and implied in the discussion that follows. The forces are generalized by the combined modes:

$$\{\Delta Q_i\} = [\chi]_i^T\ [M]\ \{\chi\}_i\ \Delta f_i \tag{6.41}$$

or

$$\{\Delta Q_i\} = [\Phi]_i^T\ [\phi]^T\ [M]\ [\phi]\ \{\Phi\}_i\ \Delta f_i \tag{6.42}$$

which, from the orthogonalization and normalization defined previously, becomes

$$\{\Delta Q_i\} = \begin{Bmatrix} f_i \\ 0 \\ 0 \\ \cdot \\ \cdot \\ \cdot \\ 0 \end{Bmatrix} \tag{6.43}$$

Therefore, only the first combined mode has a nonzero generalized force. The displacement in the first mode is

$$\Delta\xi_{1_i} = \frac{\Delta f_i}{\Omega_i^2} \tag{6.44}$$

where Ω_i is the circular frequency in the first mode for the i^{th} increment. The application of lateral force is illustrated in Figure 6.5.

The displacement in the elastic mode can be found from pseudo-mode shapes as follows:

$$\{\Delta q\}_i = \{\Phi\}_i\ \Delta\xi_{1_i} \tag{6.45}$$

The geometric displacements and accelerations at the reduced lateral coordinates are determined from the combined modal transformations:

$$\{\Delta x\}_i = \{\chi\}_i \, \Delta\xi_i \tag{6.46}$$

and

$$\{\Delta\ddot{x}\}_i = \{\chi\}_i \, \Delta f_i \tag{6.47}$$

or

$$\{\Delta\ddot{x}\}_i = \{\Delta x\} \, \Omega_i^2 \tag{6.48}$$

As described previously, displacements, accelerations, and internal loads can be evaluated for a value of Δf_i that corresponds to the elastic limit for the i^{th} increment of response.

The effect of the i^{th} incremental displacement, which accounts for the action of previous increments of force, is considered in determining the stored energy for the i^{th} increment. These previous increments of force are characterized by a different distribution:

$$E_i = \sum_{j=i}^{i} E_{ji} \tag{6.49}$$

where, for $j \neq i$,

$$E_{ji} = \Delta f_j \, \{\chi\}_j^T \, [M] \, \{\chi\}_i \, \Delta\xi_{1_i} \tag{6.50}$$

and, for $j = i$,

$$E_{ii} = \frac{1}{2} \Delta f_i \, \Delta\xi_{1_i} \tag{6.51}$$

Each term in equation (6.49) must be normalized by a different value to determine the energy in an equivalent single-degree-of-freedom oscillator. The energy for the equivalent single-degree-of-freedom oscillator for the i^{th} increment is thus

$$\bar{E}_1 = \sum_{j=i}^{i} \bar{E}_{ji} \tag{6.52}$$

where

$$\bar{E}_{ji} = \frac{\bar{E}_{ji}}{(\{x\}_i^T\{M\}) \cdot (\{x\}_j^T\{M\})} \tag{6.53}$$

A force-deflection curve for an equivalent single-degree-of-freedom system can be constructed from the increments of stored energy and the frequencies of each increment (Figure 6.6).

7. EARTHQUAKE DAMAGEABILITY

7.1 Damageability within the Response Framework

Various response parameters are determined as continuous, piecewise-linear functions of a single characteristic parameter by the quasi-static response analysis developed in the previous section. The global damageability index can be represented as a continuous function of the same characteristic parameter. A representation of the resulting damageability index is shown in Figure 7.1.

Total displacement, acceleration, and primary member loads are determined for the beginning and end of each increment of piecewise-linear structural response. Linear interpolation is used to represent the response parameters within an increment. Since primary structural members are included in the structural response analysis, their initial damage thresholds often correspond to breaks in the piecewise-linear response, i.e. to changes in stiffness. For elements that are not considered in the response analysis or for primary members for which a change in structural stiffness does not constitute the damage criterion, the level of response at which damage is initiated and at which it is irreparable must be determined. The damage demand parameter for an element is checked at the termination of each response increment to determine at which increments initial and ultimate capacities are reached. Linear interpolation is used to find the values of the characteristic parameter at initial and ultimate damage.

Since both the quasi-static response analysis and the local damageability indices are piecewise-linear approximations, a continuous piecewise-linear representation of the global damageability index is possible. Local indices are determined for all elements at the values of the characteristic parameter at which response increments terminate or at which a local index changes slope. The local indices are combined as described in section 3 to yield a global index. The global indices are plotted as functions of the characteristic parameter; the points are connected by straight lines. In Figure 7.1, the global index has been represented as a function of the normalized displacement of the fundamental elastic mode.

7.2 Predicted Damage Levels

The relationship between damageability and the characteristic response parameter determines the order in which elements in a structure are likely to be damaged as lateral force increases. The level of damageability and response must in turn be related to levels of predicted earthquake demand (see section 4). Further research is necessary before inelastic response spectra and predictions from the quasi-static response analysis described here can be fully related. However, given certain assumptions, simplifications, and with good engineering judgment, quasi-static response can be related to elasto-plastic or other response spectra. Global damageabilities can be estimated for varying magnitudes of seismic excitation with varying probabilities of occurrence from response spectra (Figure 7.2). An example of the computation involved and a description of the necessary assumptions are provided in the following sections.

Although the quasi-static response analysis has been emphasized in this report, damageability can also be assessed in conjunction with time-history analyses. The time histories of response parameters are used to determine local damageability indices in a manner analogous to that described above. Since only the highest index for each element is of interest, considerable computational effort is required to obtain a single global damageability index. The determination of suitable ground motion input and the prediction of its probability of occurrence may also be difficult. When a time-history analysis is used in conjunction with a quasi-static analysis, the results can be used to calibrate the quasi-static response predictions and corresponding damageability indices. Time-history analyses can also be used to verify predictions of response from quasi-static response analyses.

7.3 Example of a Damageability Assessment

7.3.1 Description of Model Structure

The model structure for which damageability was assessed by the procedures described above was a two-bay, three-story reinforced concrete structure designed for normal office building live loads (Figures 7.3 through 7.6). The dead load used in the design consisted of the weight of a slab, the weight of intermediate floor beams spaced at ten feet, the weight of girders, and an additional load of twenty pounds per square foot distributed over the area of each bay to account for the weight of interior partitions and/or equipment. The depth of the beams and slab was determined from deflection limits, conservatively taken as a span-to-depth ratio of fifteen for the beams and girders and twenty for the slab. Given a density of 150 pounds per cubic foot for the concrete, these values were used to determine the dead load carried by the beams. The dead load was distributed equally to the four edge beams in each bay so that each exterior beam carried a quarter of the total dead weight while the interior carried twice that weight (a quarter from each bay). A distributed live load of 60 pounds per square foot (80% office space at 50 pounds per square foot and 20% corridor space at 100 pounds per square foot) was assumed and load factors of 1.4 for the dead load and 1.7 for the live load were used. For the design of the roof beams, the loading was decreased since there was no partition on the roof and the live load was omitted. The design ultimate loads are indicated on the elevations in Figures 7.4 through 7.5.

A lateral load of 25.5 pounds per square foot of cladding (15 psf multiplied by a load factor of 1.7) was included to account for lateral loading due to wind; it was assumed that the design for this lateral load would provide adequate earthquake resistance. The design moments were calculated using factors specified in ACI 318-71 , Section 8.4.2, and from these the required steel areas were determined for the beams. Top and bottom bars were considered to run the length of the beams, thereby ensuring uniform stiffness and improved resistance to moment reversal and fire. The cross sections are illustrated in Figure 7.7. Where the steel areas were similar for different beams, one section was designed to withstand the greatest moment. Shear reinforcement was not explicitly designed, nor included in the stiffness calculations, but was considered to have been provided in sufficient quantity to ensure that plastic hinges would form under dynamic loading.

The columns were designed to withstand the axial load and moment exerted at the base of the member. All corner columns were identical, symmetrically reinforced sections (Figure 7.8). The intermediate columns were reinforced differently in the two directions (Figure 7.9).

Given these beam and column sections, the effective stiffnesses were calculated for cracked, transformed cross sections. Moment capacities of the beams were determined; the double reinforcement was considered. Interaction diagrams were drawn for two column sections (Figures 7.8 and 7.9). Biaxial effects were ignored; stiffness and moment capacity of the asymmetrical center columns were calculated for each direction. Axial stiffness of the gross concrete cross section was evaluated.

Steel yield strength f'_y was assumed to be 60 ksi and concrete design strength f'_c was assumed to be 4 ksi. The modulus of elasticity of the concrete was calculated to be 57,000 $\sqrt{f'_c}$ or 3.6×10^6 psi; a steel modulus of 29×10^6 psi was assumed. The resulting stiffnesses and capacities are noted next to the sections shown in Figures 7.7 to 7.9.

7.3.2 Structural Modeling and Response Analysis

The model structure was analyzed for quasi-static planar response for the two-bay y-direction (Figure 7.10). Since the front and rear frames of the structure were identical, only one frame was analyzed. Structural response at nine nodal points, one node at each beam and column connection, was calculated. Each node included two translational coordinates and one rotational coordinate; however, the lateral motion at all nodes on a given floor was represented by response at a single y-coordinate.

The stiffness matrix for the structure was assembled from the stiffnesses of beam and beam-column members. The member stiffnesses were calculated using properties for cracked sections and centerline-to-centerline spans. Inelastic response was assumed to occur only at the ends of members (nodes). Each beam or column member was modeled as an elasto-plastic element in parallel with an elastic element in order to represent bilinear stiffness. For this example, the stiffness of the elastic element was assumed to be one percent of the initial stiffness of the parallel elasto-plastic element. Geometric stiffness or P-Δ effects were neglected.

Since the shape of the lateral force distribution was assumed to remain constant throughout the analysis, the equations developed in section 6.3 were valid. The response analysis was performed using the computer program EPRESP (see Appendix). The gravity loads consisted of the full dead load and an additional load of 20 pounds per square foot. The loads were assumed to be distributed uniformly on the beams. The loads on the frame are shown in Figure 7.10. The loads applied at the nodes consisted of fixed-end moments and shear loads. In calculating the internal load at the end of a beam, the fixed-end moment was considered to be applied on the member side, a potential hinge location.

The energy-normalized mode shapes and frequencies are shown in Figure 7.11. The shape factors of the resulting lateral force distributions were 0.195, 0.409, and 0.396 for the first, second, and third floors, respectively. In the response analysis, the magnitude of the force distribution was increased quasi-statically. The formation of a plastic hinge in an elasto-plastic element was considered to terminate a response increment. The order of hinge formation is illustrated in Figure 7.12. Although the hinge locations taken together constitute a potential collapse mechanism, the residual stiffness of the parallel elastic members would prevent collapse.

As noted in section 6, quasi-static response can be represented by single values of force and displacement. The force-displacement relationship for this example is plotted in Figure 7.13, where the first set of scales is the energy-normalized modal displacement q and the generalized force magnitude f, and the second set of scales is the displacement $\bar{q}$ and force $\bar{f}$ of an equivalent single-degree-of-freedom system. The potential energy of the structure, or the equivalent system, is represented by the area under the curve.

7.3.3 Damageability Indices

The damageability of a structure is determined from the damageability of elements in a damage model of the structure. Damage elements include primary and secondary structural elements. Primary elements are also elements in the structural model. In this example, each beam and column in the frame was a damage element. Secondary elements are all other elements for which damage is considered. For this example, the only secondary elements considered were the six interior partitions of gypsum board on metal studs, one partition for each bay and story. Since the frame represents one-half of the structure, however, damageability was assessed for three partitions only, one for each story.

The local damageability indices were determined for each element as a function of a single response parameter. For the partition elements, damageability was assumed to be a function of interstory drift [18]. Damage was assumed to be initiated at a story drift angle of 1/125 for the gypsum board on metal studs. Damage that would require complete replacement was estimated to occur at a drift of 1/30. The resulting profile of the local damageability index is shown in Figure 7.14. The index is approximated to be piecewise linear as was described in section 3.

The damageability index for a beam-column is determined by two criteria. The response analysis is examined to determine if a plastic hinge has formed; if so, damage is assumed to have been initiated. The extent of damage is determined from the ductility factor; initial damage is assumed to occur at a factor of one. The ductility factor at which a beam-column will fail depends on many factors, including loading history and detailing of reinforcement. Three maximum ductility factors and corresponding damageability indices were considered; while the local damageability index is one, the collapse of a column is also considered catastrophic to a structure.

The ductility factor for beam and column hinges is determined from the curvature of the elastic element in parallel with the hinged elasto-plastic element. The local damageability index for the central first-story column is shown in Figure 7.15. Maximum allowable ductility factors of three, five, and ten are represented in the figure.

Each element in the damage model is assigned an importance factor p in order to establish the global damageability index. The partitions were assigned an importance factor of 1.0, and the beams and columns factors of 3.0 and 5.0, respectively; the higher importance factors assigned to the latter two elements reflect the greater importance of these structural elements. The global damageability index was calculated by summing the weighted local indices and then dividing the result by the sum of the importance factors (equation (3.5)). The failure of a column is considered catastrophic for the structure as a whole; the global damageability index is thus set equal to one when a column fails. The global damageability index for the model structure is shown in

Figure 7.16 as a function of the displacement of the equivalent single-degree-of-freedom system. The three curves in that figure represent the response of the beam-columns for the three maximum allowable ductility factors considered in the analysis. In Figure 7.17 the index is shown as a function of lateral roof displacement.

7.3.4 Predicted Response and Damageability

An elasto-plastic response spectra proposed by Newmark [15] was used to estimate the maximum displacement of the single-degree-of-freedom system. The global damageability index that corresponds to predicted displacement was determined from the damageability profiles that were described in section 7.3.3. The response spectra is reproduced in Figure 7.18. A fundamental mode frequency of 0.83 cps was used; the maximum displacement was estimated to be 5.3 inches. The frequency 0.83 cps falls within a region of the spectra for which the energy of an elasto-plastic system can be approximated by that of an elastic system with the same frequency. Therefore, the maximum elasto-plastic displacement was related to the elastic displacement by a multiplier, $\mu/\sqrt{2\mu-1}$, where μ is the ductility factor of the elasto-plastic system. This multiplier was used to construct a force-displacement relationship for the elasto-plastic system given an initial frequency, a maximum elastic displacement (from an elastic response spectrum), and a ductility factor.

The resulting force-displacement relationship for the model structure is shown in Figure 7.19 along with the curves from Figure 7.13. The scales are for the equivalent single-degree-of-freedom system. Although the curve is piecewsie linear and not elasto-plastic, the maximum displacement of the equivalent system was estimated to be between that for an inelastic system and that for an elasto-plastic system with a ductility factor of 2. A maximum displacement of 5.8 inches was estimated. The global damageability factor that corresponds to this displacement is determined from Figure 7.16. The vertical line marked 'Spectra 1' represents a displacement of 5.8 inches. Global damageability indices for the system are therefore 0.14, 0.08, and 0.04 for maximum beam-column ductility factors of 3, 5, and 10, respectively. The damageability indices indicate that the structure would suffer moderate to minor damage, the degree of damage depending on the allowable beam-column ductility.

A hypothetical elastic spectra was assumed to produce a maximum displacement of 10.0 inches for a frequency of 0.83 cps. As before, this point was assumed to lie on a portion of the response spectrum for which the energy of an elasto-plastic system is identical to that for an elastic system with the same initial frequency. Force-displacement relationships were calculated and are presented with the curve for the structure in Figure 7.20. The maximum displacement of the equivalent system falls between the displacements of elasto-plastic systems with ductility factors of 3 and 4 (Figure 7.20). A maximum displacement of 14.5 inches was estimated. The vertical line marked 'Spectra 2' in Figure 7.16 corresponds to this displacement.

The resulting global damageability factors for the structure were 1.0, 0.37, and 0.19 for allowable beam-column ductilities of 3, 5, and 10, respectively. For a maximum ductility factor of 3, the structure was therefore predicted to be damaged beyond repair and to suffer at least partial collapse. For a factor of 5, the structure was predicted to suffer severe damage and to be close to partial collapse. For a factor of 10, only moderate damage was predicted.

8. CONCLUDING REMARKS

The concept of global damageability indices for loadings of increasing intensity and of cumulative damageability indices for a prescribed set of loadings leads to a number of interesting applications. Such procedures need not be limited to the evaluation of earthquake damageability; the effects of other natural hazards can also be evaluated. These procedures also need not be limited to the evaluation of existing buildings; they can be used to assess the performance of a proposed design, or to evaluate code requirements and particularly the effects of proposed changes in codes on the overall damageability of structures.

More specifically, damageability indices for increasing load intensity provide the following information:

1) a measure of load levels at the initiation of damage;
2) a measure of load levels at acceptable damage levels.
3) a range of load levels at which a structure will not be sensitive to damage to a small number of critical elements;
4) a means of identifying the most critical elements which influence damageability in a structural system;
5) a means of identifying potential cumulative effects of damage;
6) a measure of the effectiveness of different partial hazard abatement measures such as strengthening of only the most vulnerable elements.

While further calibration studies are needed before values of importance factors and capacity bounds can be reliably established, the procedures set forth in this report are a significant step toward the development of a reliable means of assessing the damageability of existing and proposed structures.

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APPENDIX

INSTRUCTIONS FOR USE OF THE COMPUTER PROGRAM EPRESP

EPRESP is a computer program developed to perform a piecewise-linear, quasi-static structural analysis of idealized two-dimensional rectangular frames. Earthquake inertial forces are approximated by a distribution of lateral force such that the lateral response of the structure can be expressed in terms of response at a single coordinate, the fundamental mode of the initially elastic structure. The magnitude of lateral force distribution is increased in linear steps, each step terminating with the formation of a plastic hinge in a beam or column element. The analysis is complete when a collapse mechanism forms or when all potential plastic hinging has occurred.

Static gravity loads are applied to the structure prior to the application of lateral earthquake forces. Distributed beam loads are applied as concentrated shears and fixed-end moments; however, in computing internal bending moments at the ends of beam elements, the fact that the applied moments are due to distributed loads is considered.

Structural Idealization

A structure is idealized as a set of nodes connected by beam-column elements. The stiffness of an element is represented by three displacements at each end; however, element coordinates can be constrained by or collocated with the coordinates of other elements if desired. The elements can be made inextensible by properly specifying local-to-global coordinate transformation.

All nodes and elements are numbered by the user; however, all coordinates at which inertial force will be applied must be ordered first. The lateral motion of all nodes at a given floor should share a coordinate and lateral force can be applied at these coordinates only.

Beam-column members are idealized as bilinear, two-component, parallel elements. The elasto-plastic component can form plastic hinges at the ends only. The transformation from a global set of nodal coordinates to a single modal coordinate is calculated and applied automatically. Although the motion of the structure can be characterized by that of a single coordinate, the response parameters throughout the structure are calculated at the end of each response increment.

Program Information

The computer program is written in Fortran IV and was developed for use on a CDC 6400 series computer. Most required core storage is established in a partitioned array in blank COMMON. Only dimension sizes and integers that specify the dimension sizes need be changed to accommodate different structures. The program implements a series of subroutines that perform specific tasks. Many of these subroutines have been adopted from the program described in reference 19.

Input Information

A. TITLE CARDS (12A6) 3 cards

Columns 1 - 6: STOP --, if no more structures are to be input.

Columns 1 - 72: Information must be printed at the top of pages of output.

B. OPERATION CONTROL SPECIFICATIONS (6(A3,I2)) 1 card

Columns 4 - 5: NC, number of nodal coordinates

Columns 9 - 10: NE, number of bilinear beam-column elements

Columns 14 - 15: NM, number of lateral coordinates; also, the size of the mass matrix and the number of nodes

Columns 19 - 20: NG, number of ground springs; can be zero

Columns 24 - 25: NN, number of nodes

Columns 29 - 30 IFPRNT = 1, if additional output is desired; IFPRNT = 0, if regular output is desired.

C. DEGREE-OF-FREEDOM TABLE

The number of cards is determined by input in columns 1 - 5.

Columns 1 - 5: NODE = 00000 if no more cards are to be input for the table; otherwise, a node identifier.

Columns 6 - 10: x-, y-, z-, θx-, θy-, and θz-global coordinate numbers for the node

11 - 15:

16 - 20:

21 - 25:

26 - 30:

Columns 36 - 45: x-, y-, and z-locations of the node

46 - 55:

56 - 65:

Columns 67 - 77: Comment to be printed with node information.

D. <u>ELEMENT SPECIFICATION</u> NE sets of 3 cards for each element

1. <u>Element Properties</u> (10X,5E10.c)

Columns 11 - 20: EI, bending stiffness of member

Columns 21 - 30: AE, axial stiffness of member

Columns 31 - 40: AL, length of member

Columns 41 - 50: AB1, length of rigid link on first end

Columns 51 - 60: AB2, length of rigid link on second end

2. <u>Element Capacities</u> (10x,5E10.3) 1 card

Columns 11 - 20: TM1, maximum elastic moment at the first end; hinging in the elasto-plastic component begins when this total capacity has been reached.

Columns 21 - 30: TM2, maximum elastic moment of second end

Columns 31 - 40: P, the fraction of the member that is to remain elastic after hinge transformation

Columns 41 - 50: FEM1, fixed-end moment applied at first end (also included with gravity loads)

Columns 51 - 60: FEM2, fixed-end moment at second end

3. Element-to-Global Transformation Vector (6I5) 1 card

The order of the numbers on this card corresponds to the element coordinate system. The numbers indicate the global coordinates to which the local coordinates correspond.

E. GROUND SPRING STIFFNESS VALUES (10x,7E10.3)

If NG = 0, no card is read; the ground spring values are input in order, seven per card, until all NG values have been read.

F. GROUND SPRING COORDINATE NUMBERS (16I5)

If NG = 0, no card is read; the numbers are input in the same order as the stiffness values until all NG values have been input.

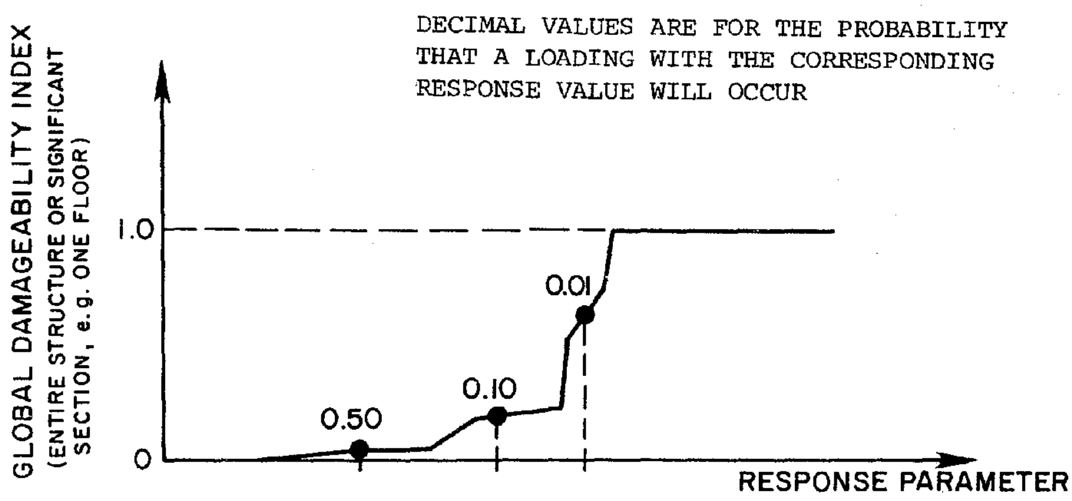


FIGURE 2.1 TYPICAL DAMAGEABILITY PROFILE FOR A CONTINUOUS QUASI-STATIC ANALYSIS

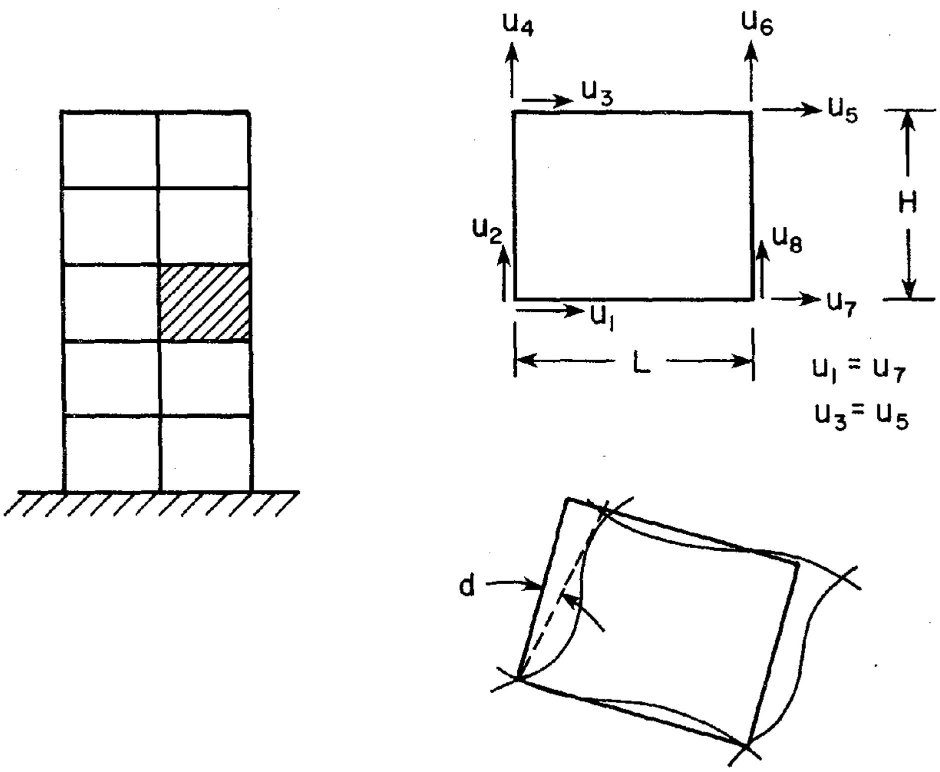
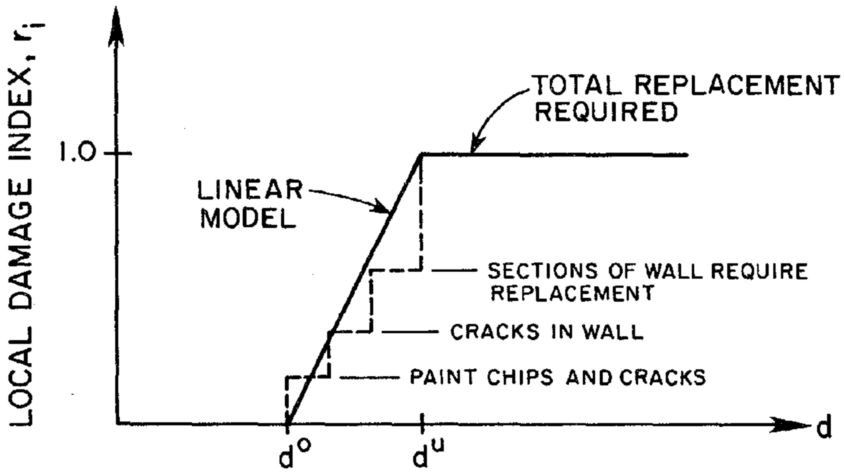


FIGURE 3.1 DEMAND PARAMETER FOR AN INFILLED WALL



d^o - the value of average tangential drift at which damage is initiated

d^u - the value of average tangential drift at which a wall becomes irreparably damaged and must be replaced

FIGURE 3.2 TYPICAL LOCAL DAMAGE PROFILE FOR AN INFILLED WALL

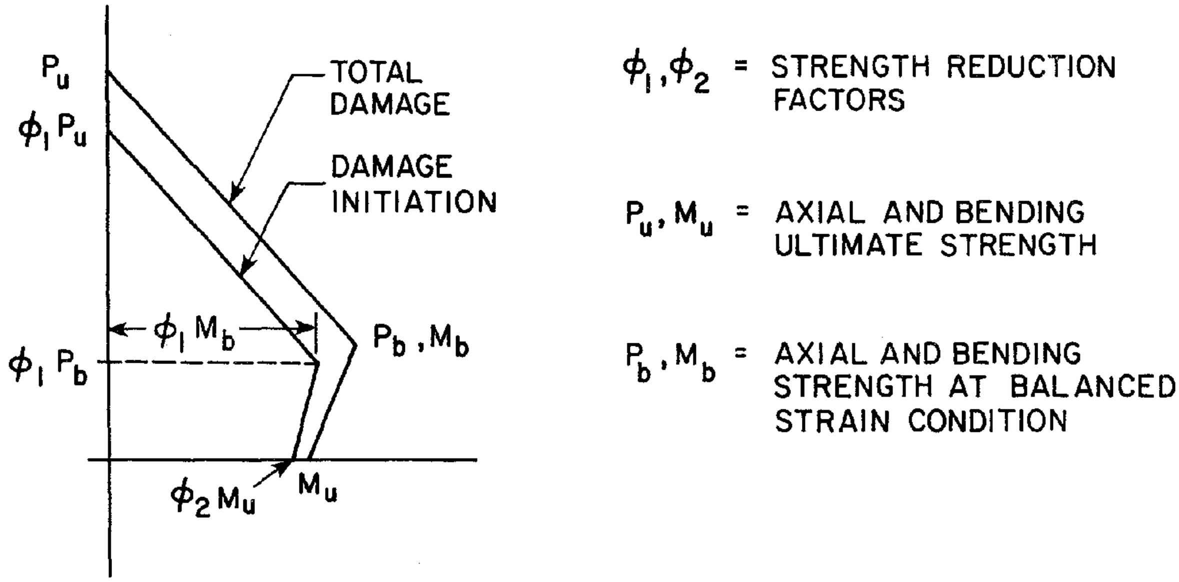


FIGURE 3.3 STRENGTH PARAMETERS FOR DAMAGE EVALUATION OF A COLUMN

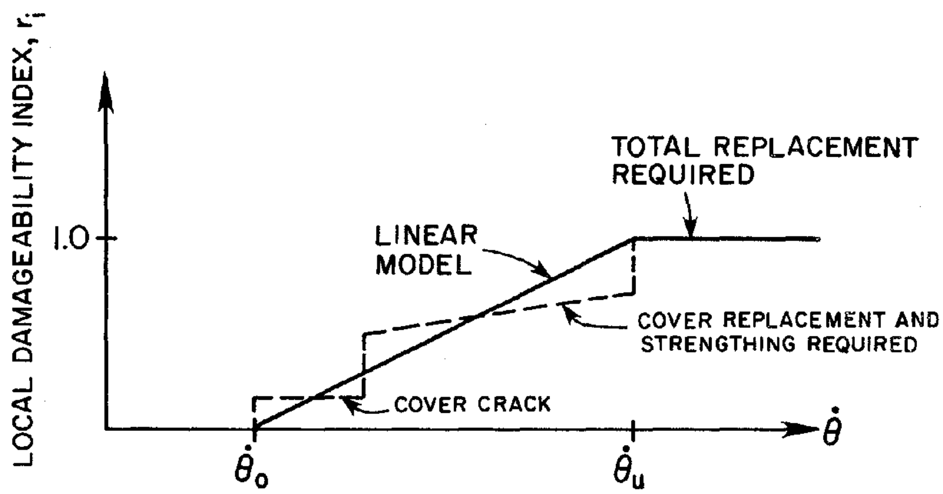


FIGURE 3.4 PARAMETER FOR DEGREE OF DAMAGE OF A COLUMN

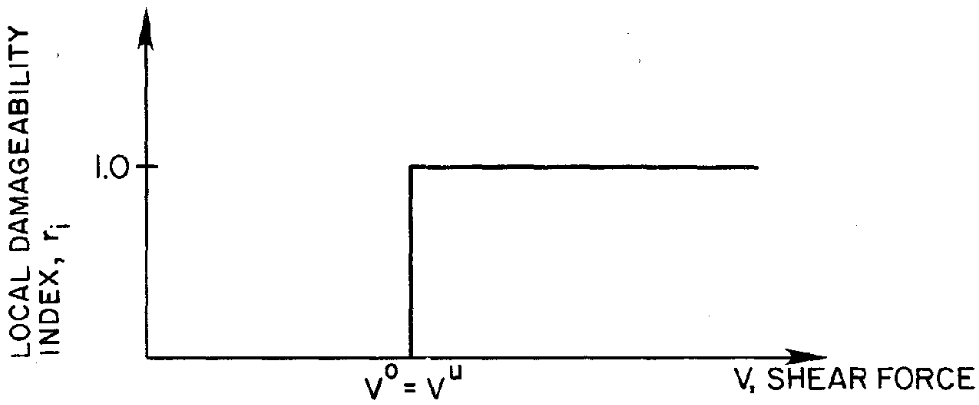


FIGURE 3.5 PARAMETER FOR ASSESSMENT OF DAMAGE DUE TO SHEARING FORCE IN A COLUMN

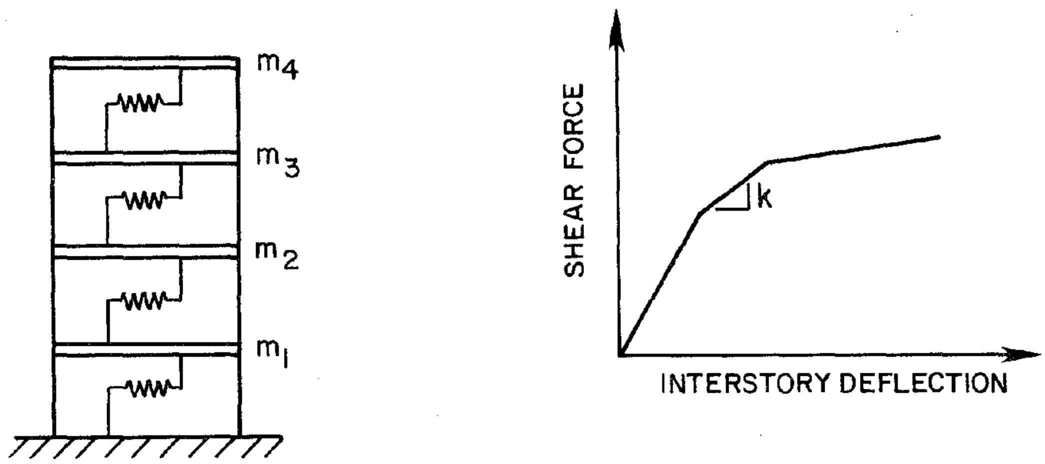


FIGURE 4.1 TYPICAL MODEL FOR A FIRST SCREENING

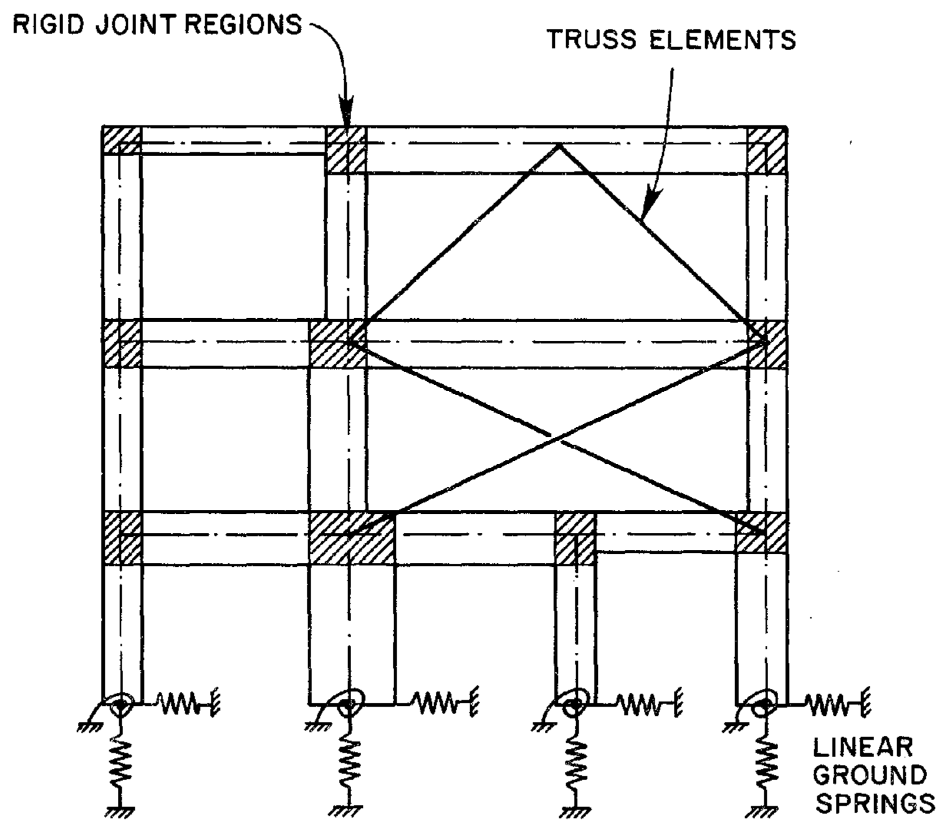


FIGURE 4.2 MODEL FOR AN EARLY SCREENING

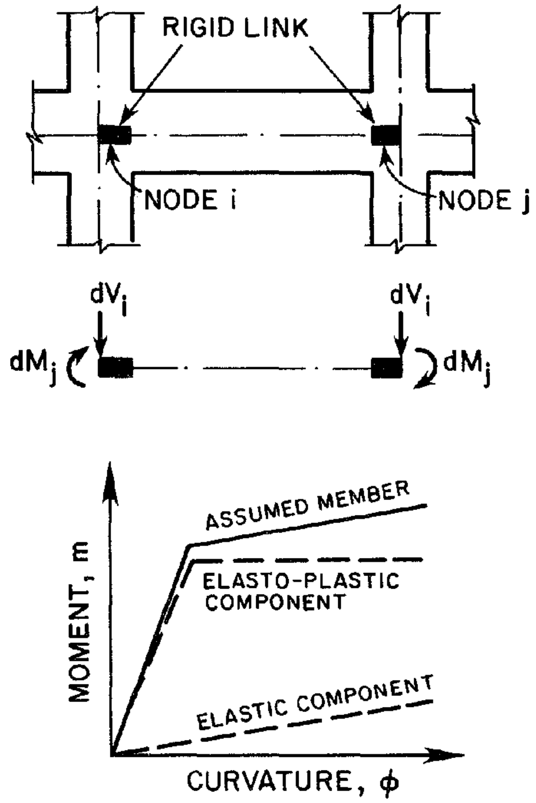


FIGURE 4.3 TYPICAL BEAM MODEL

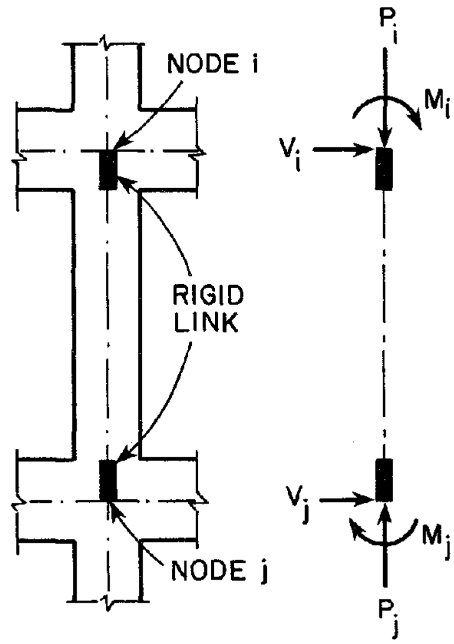


FIGURE 4.4 TYPICAL COLUMN MODEL

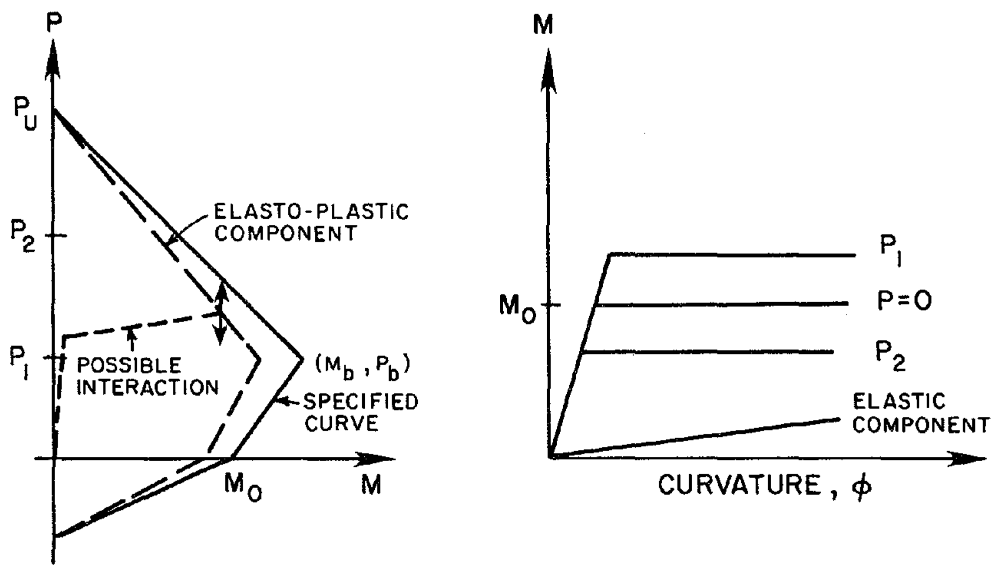


FIGURE 4.5 YIELD CRITERIA FOR A COLUMN ELEMENT

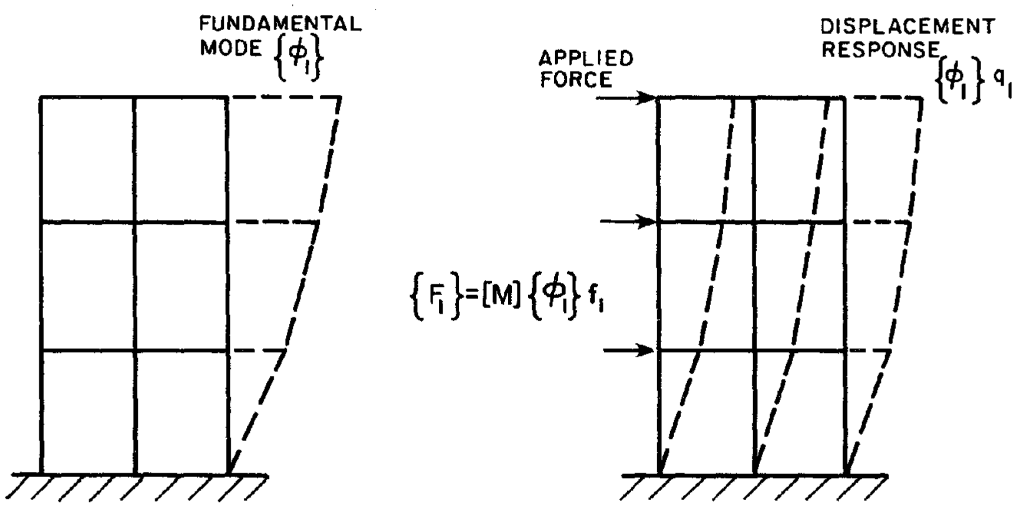


FIGURE 6.1 LINEAR ELASTIC QUASI-STATIC RESPONSE

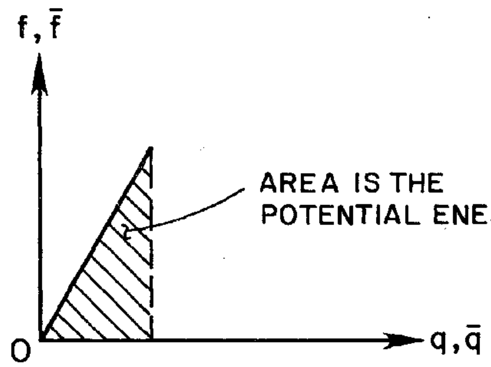


FIGURE 6.2 FORCE-DISPLACEMENT RELATIONSHIP FOR ELASTIC RESPONSE

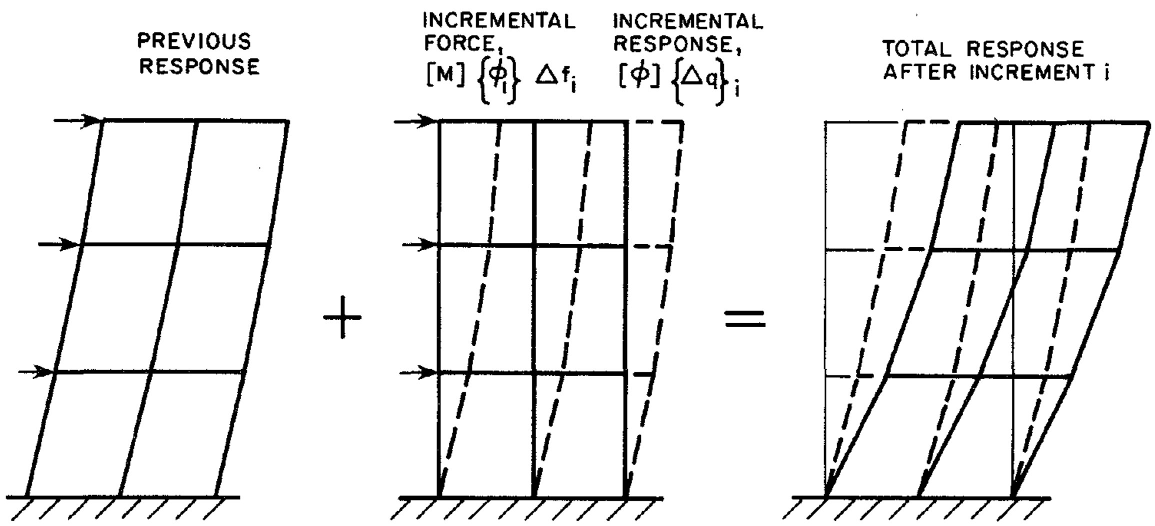


FIGURE 6.3 INCREMENT OF NONLINEAR RESPONSE WITH FORCE DISTRIBUTION OF CONSTANT SHAPE

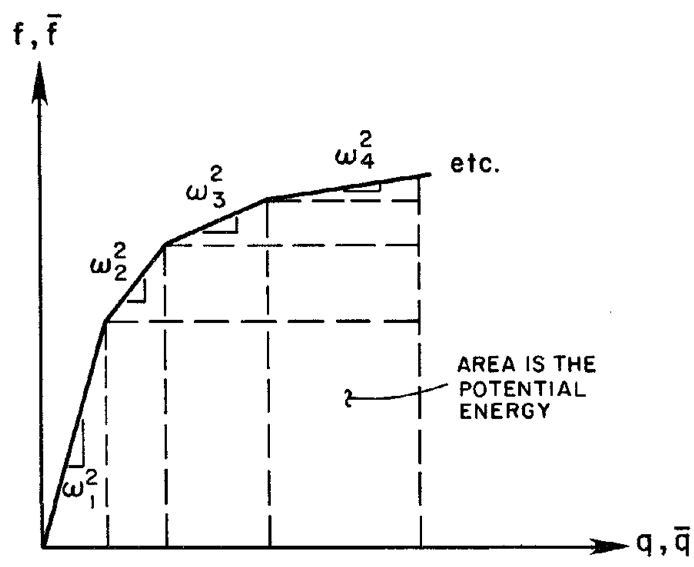


FIGURE 6.4 FORCE-DISPLACEMENT RELATIONSHIP FOR NONLINEAR INCREMENTS WITH A FORCE DISTRIBUTION OF CONSTANT SHAPE

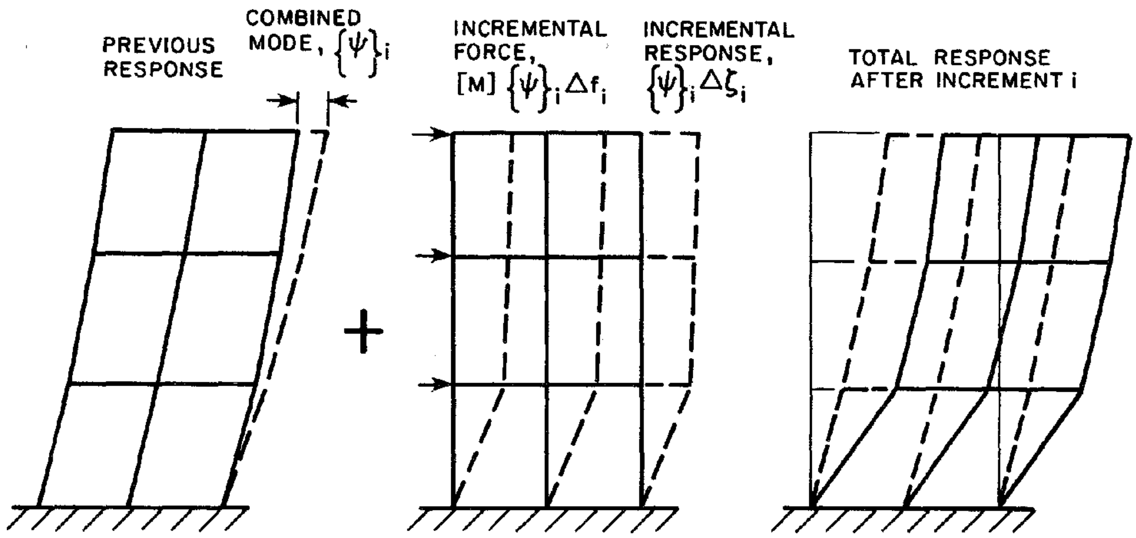


FIGURE 6.5 INCREMENT OF NONLINEAR RESPONSE WITH MODIFIED FORCE DISTRIBUTION

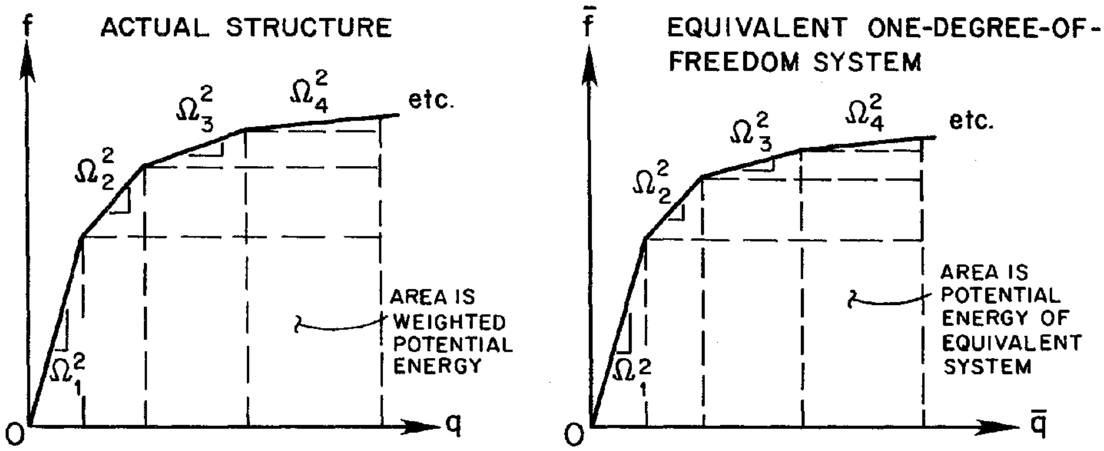


FIGURE 6.6 FORCE-DISPLACEMENT RELATIONSHIPS FOR NONLINEAR INCREMENTS WITH MODIFIED FORCE DISTRIBUTIONS

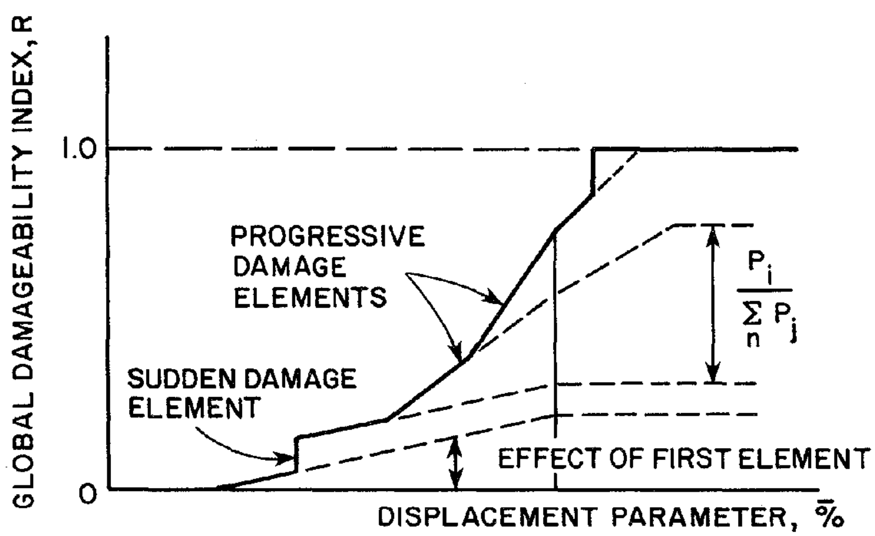


FIGURE 7.1 GLOBAL DAMAGEABILITY FOR A QUASI-STATIC RESPONSE ANALYSIS

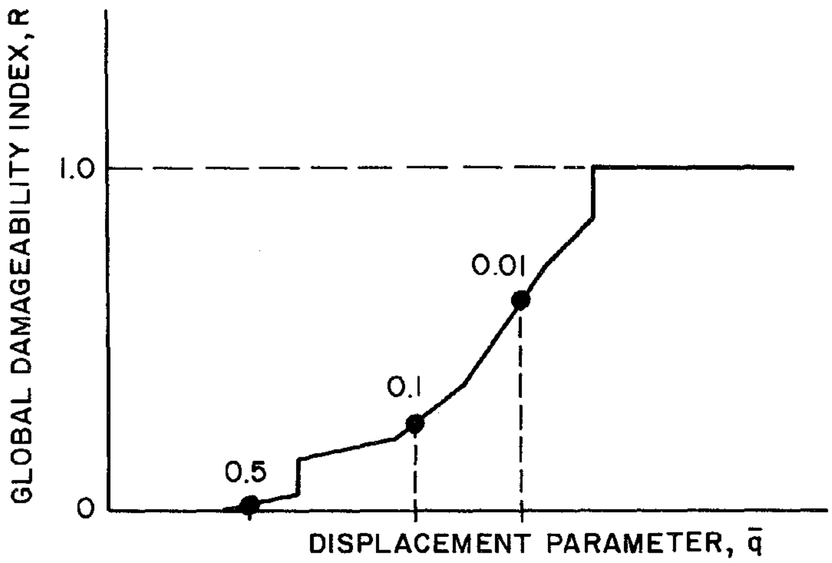


FIGURE 7.2 DAMAGEABILITY FOR EARTHQUAKES OF VARYING PROBABILITY OF OCCURRENCE

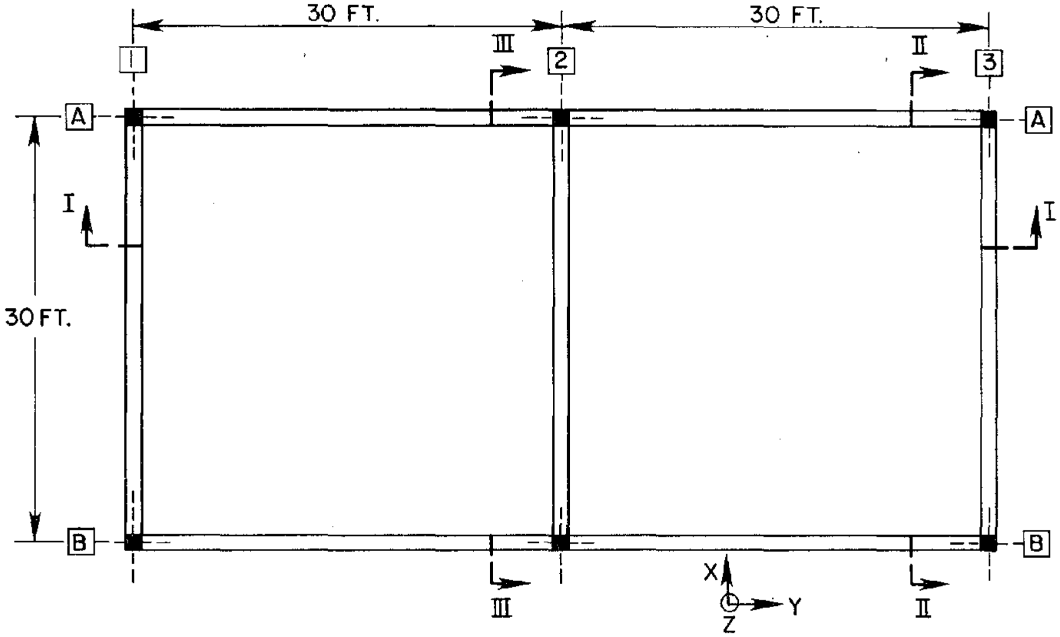


FIGURE 7.3 PLAN VIEW OF MODEL STRUCTURE

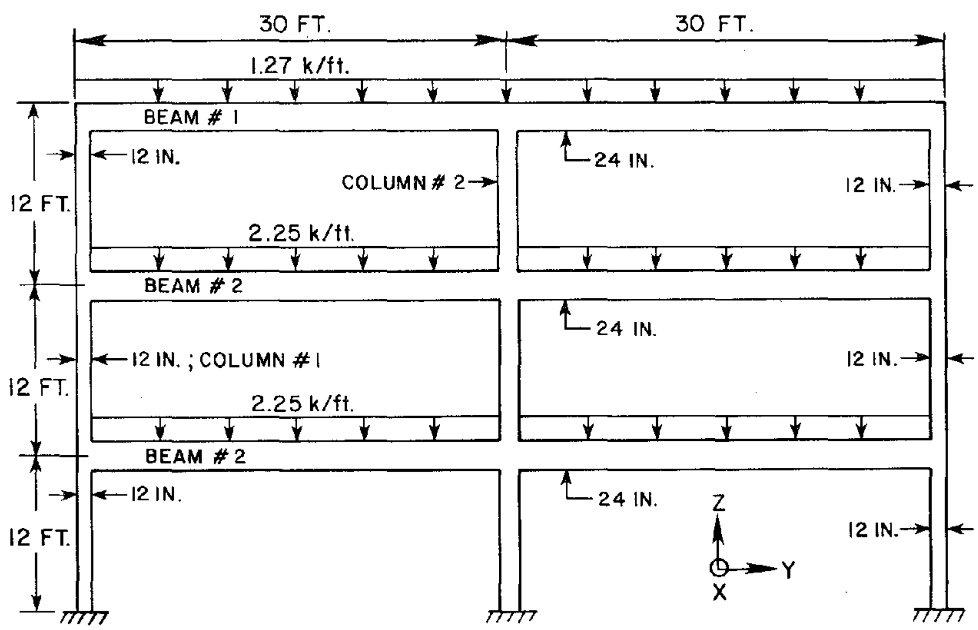


FIGURE 7.4 SECTION I-I OF MODEL STRUCTURE
(X-Y DIRECTION)

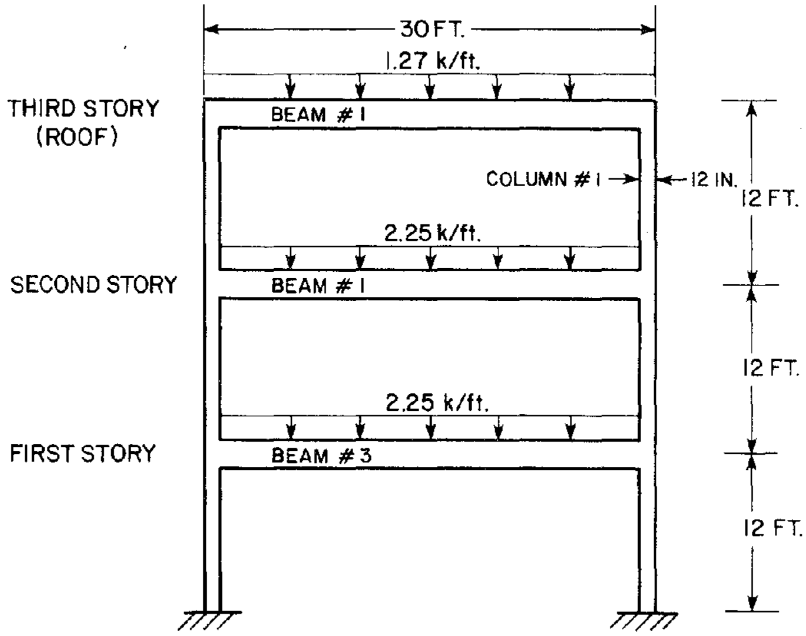


FIGURE 7.5 EXTERIOR XX FRAME OF MODEL STRUCTURE, SECTION II-II

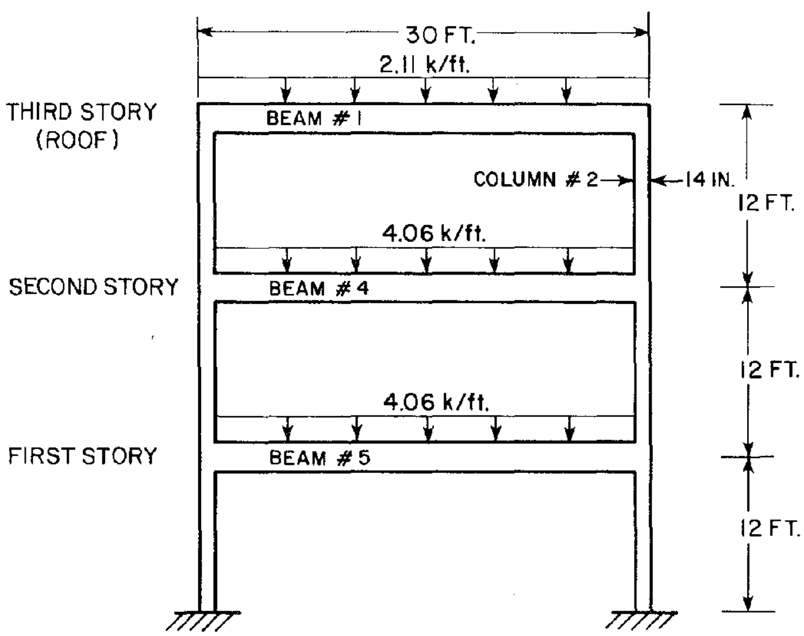
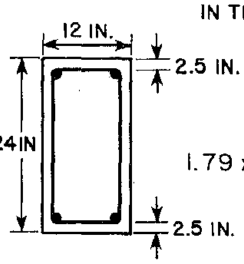
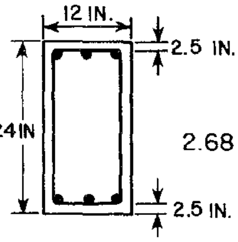
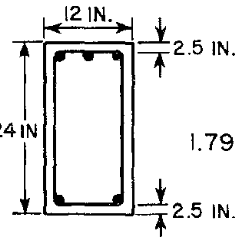
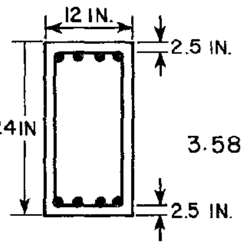
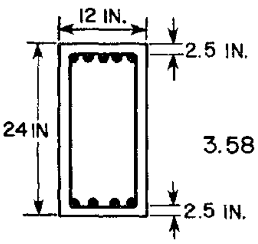


FIGURE 7.6 INTERIOR XX FRAME OF MODEL STRUCTURE, SECTION III-III

BEAM NO.	RE. BAR LOCATION	POSITIVE MOM. CAP. (BOTTOM STEEL IN TENSION)	NEGATIVE MOM. CAP. (TOP STEEL IN TENSION)	E I. (CRACKED SECTION I)	A E (GROSS X-SECTION OF CONCRETE)
1		1.79×10^6 lb-in^2	1.79×10^6 lb-in^2	1.44×10^{10} lb-in^2	1.04×10^9 lb
2		2.68×10^6 lb-in^2	2.68×10^6 lb-in^2	2.03×10^{10} lb-in^2	1.04×10^9 lb
3		1.79×10^6 lb-in^2	2.68×10^6 lb-in^2	1.46×10^{10} lb-in^2	1.04×10^9 lb
4		3.58×10^6 lb-in^2	3.58×10^6 lb-in^2	2.58×10^{10} lb-in^2	1.04×10^9 lb
5		3.58×10^6 lb-in^2	4.57×10^6 lb-in^2	2.62×10^{10} lb-in^2	1.04×10^9 lb

*ALL LONGITUDINAL REINFORCING BARS ARE #8

FIGURE 7.7 BEAM TYPES, CROSS SECTION, STIFFNESSES, AND CAPACITIES OF MODEL STRUCTURE

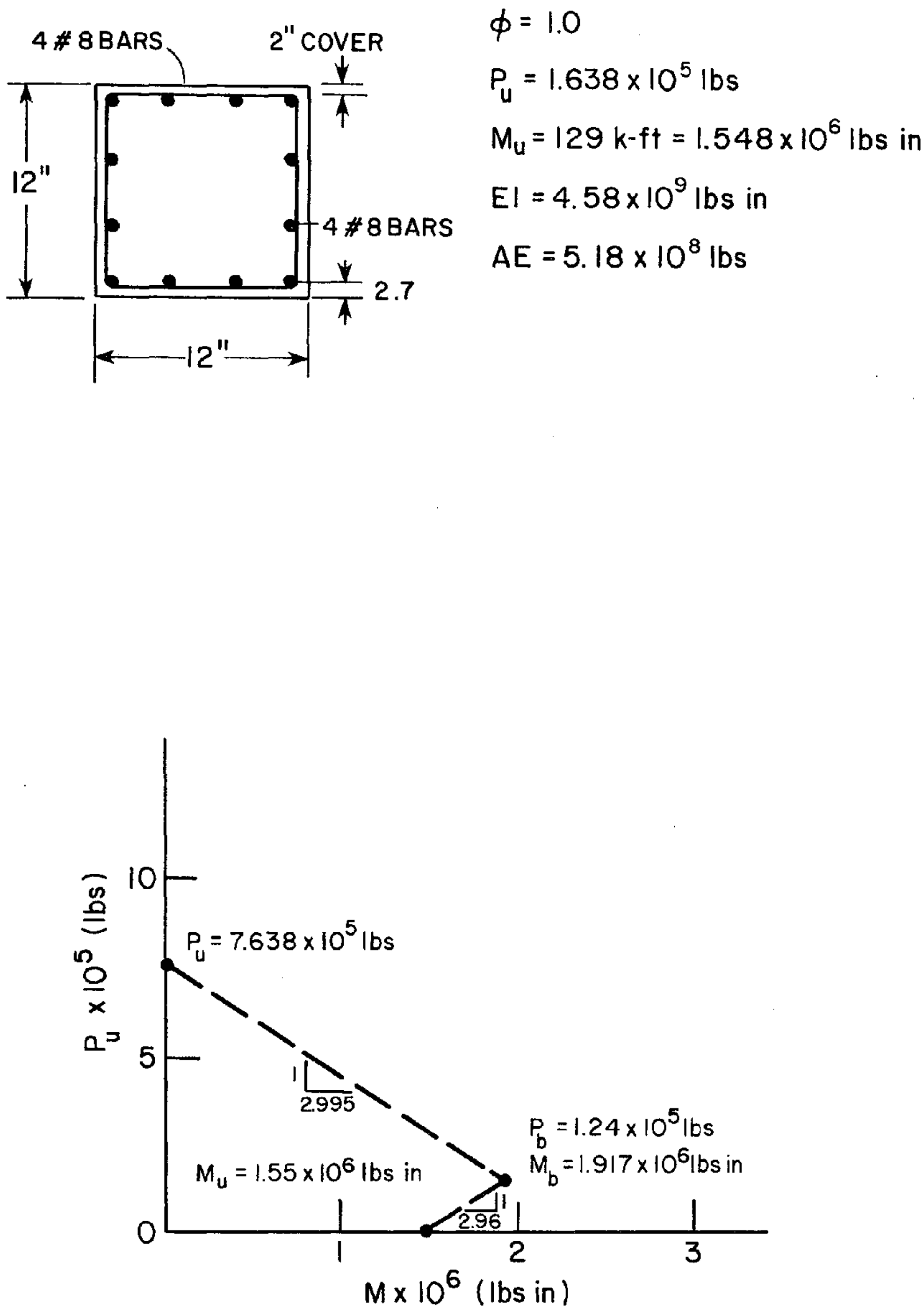


FIGURE 7.8 COLUMN NUMBER 1 (CORNER)

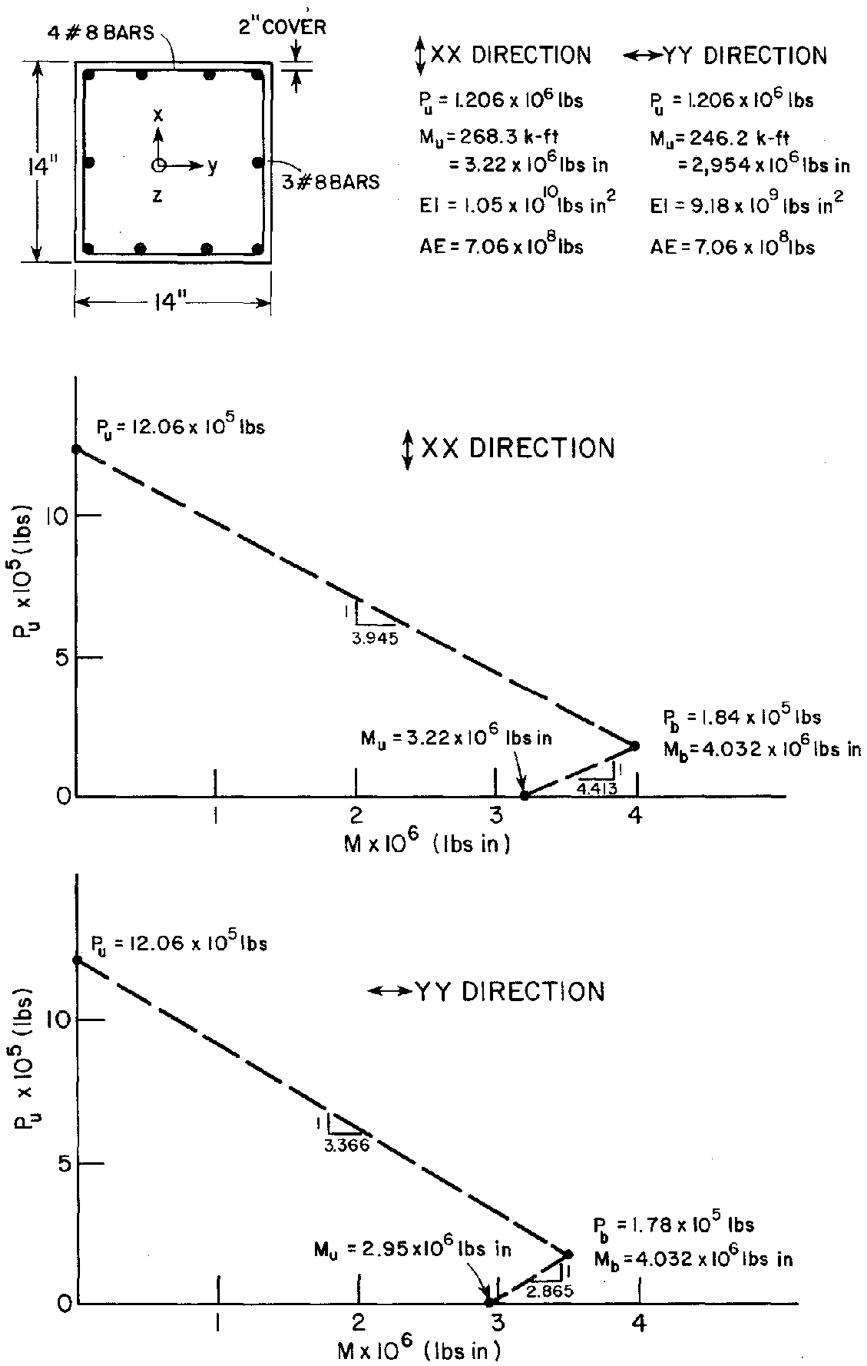


FIGURE 7.9 COLUMN NUMBER 2 (CENTER)

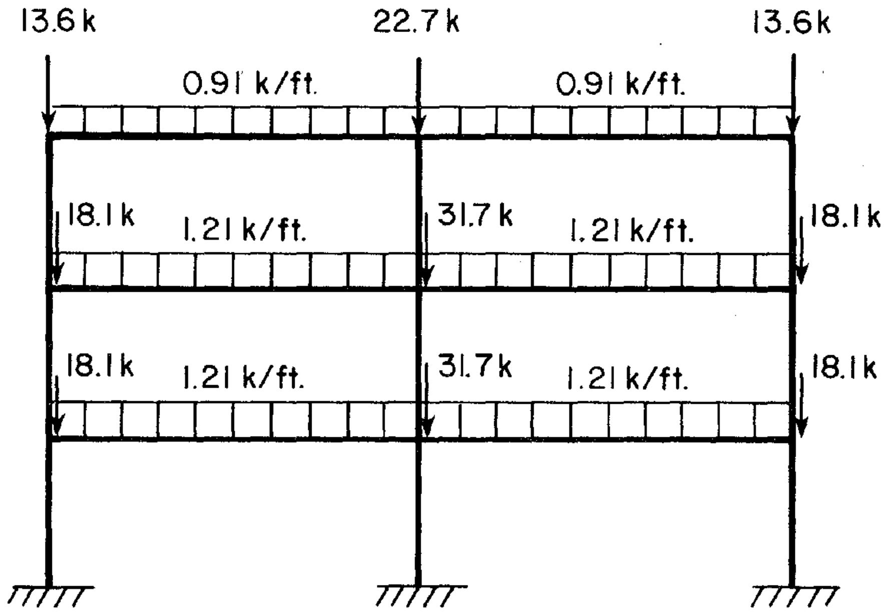


FIGURE 7.10 GRAVITY LOAD DISTRIBUTION

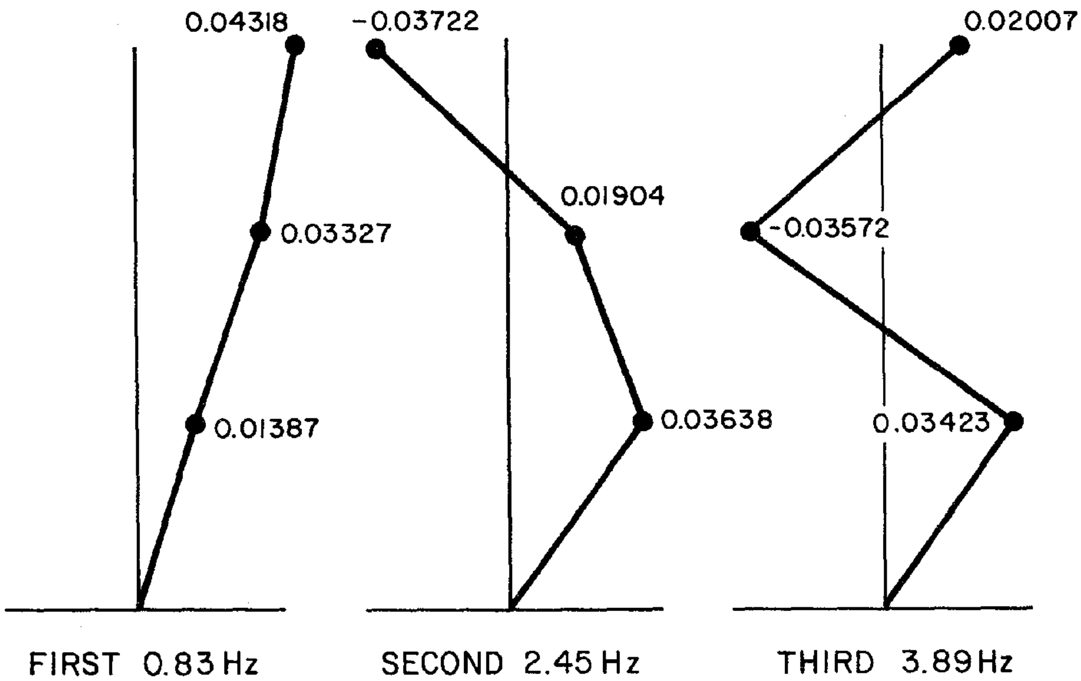


FIGURE 7.11 ENERGY-NORMALIZED MODES

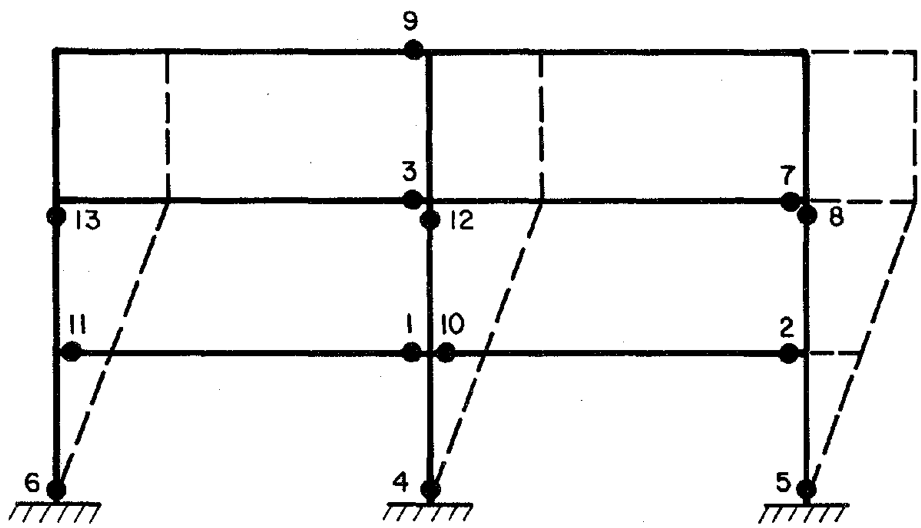


FIGURE 7.12 ORDER OF HINGES AND COLLAPSE MECHANISM

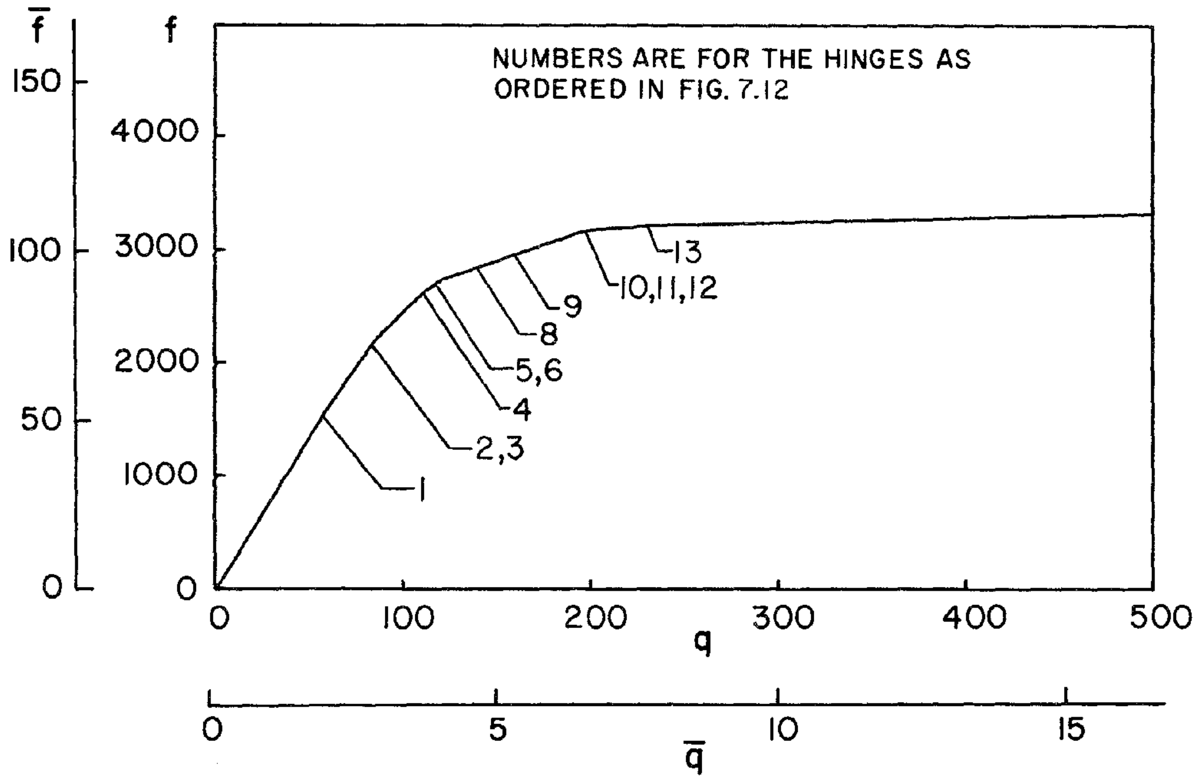


FIGURE 7.13 FORCE-DISPLACEMENT RELATIONSHIP

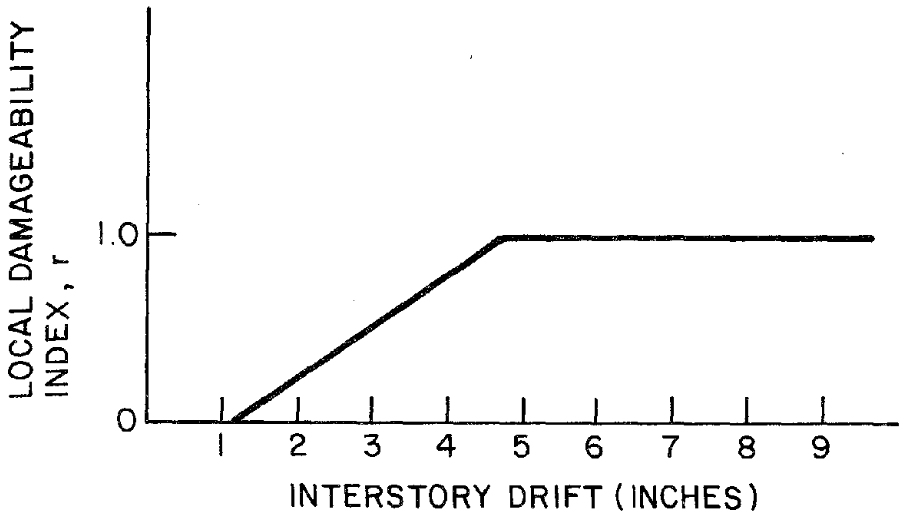


FIGURE 7.14 DAMAGEABILITY INDEX FOR PARTITIONS

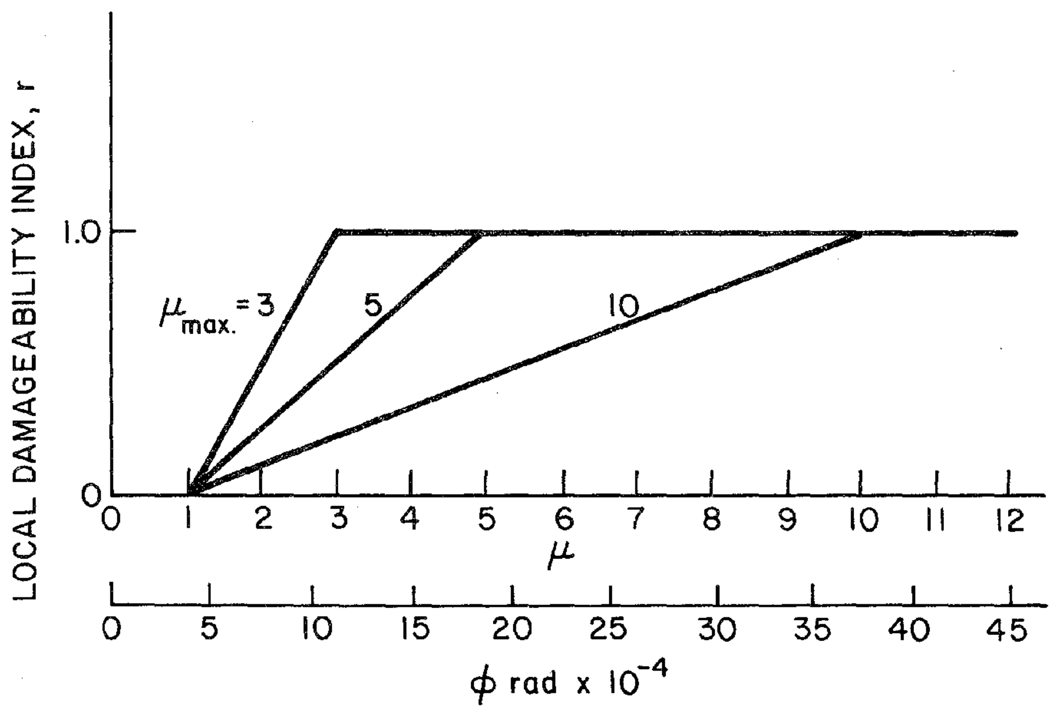


FIGURE 7.15 DAMAGEABILITY INDEX FOR THE FIRST STORY, CENTER COLUMN

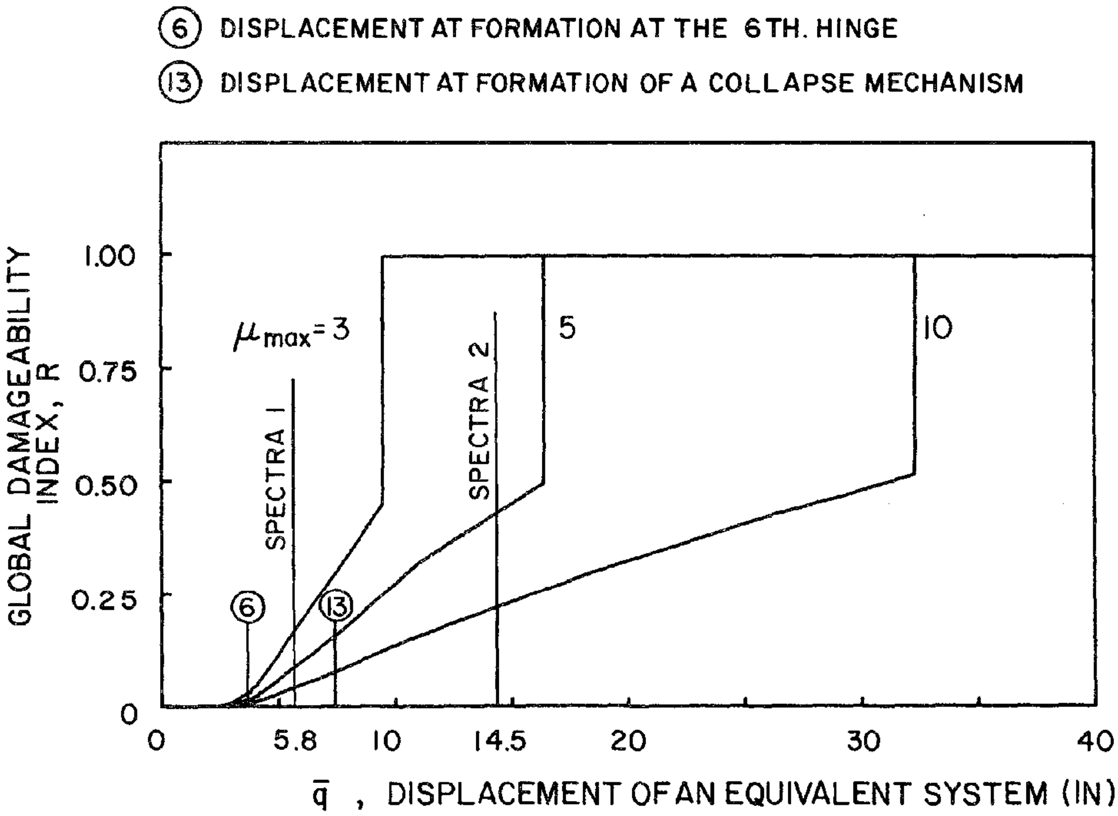


FIGURE 7.16 GLOBAL DAMAGEABILITY INDEX AND EQUIVALENT DISPLACEMENT

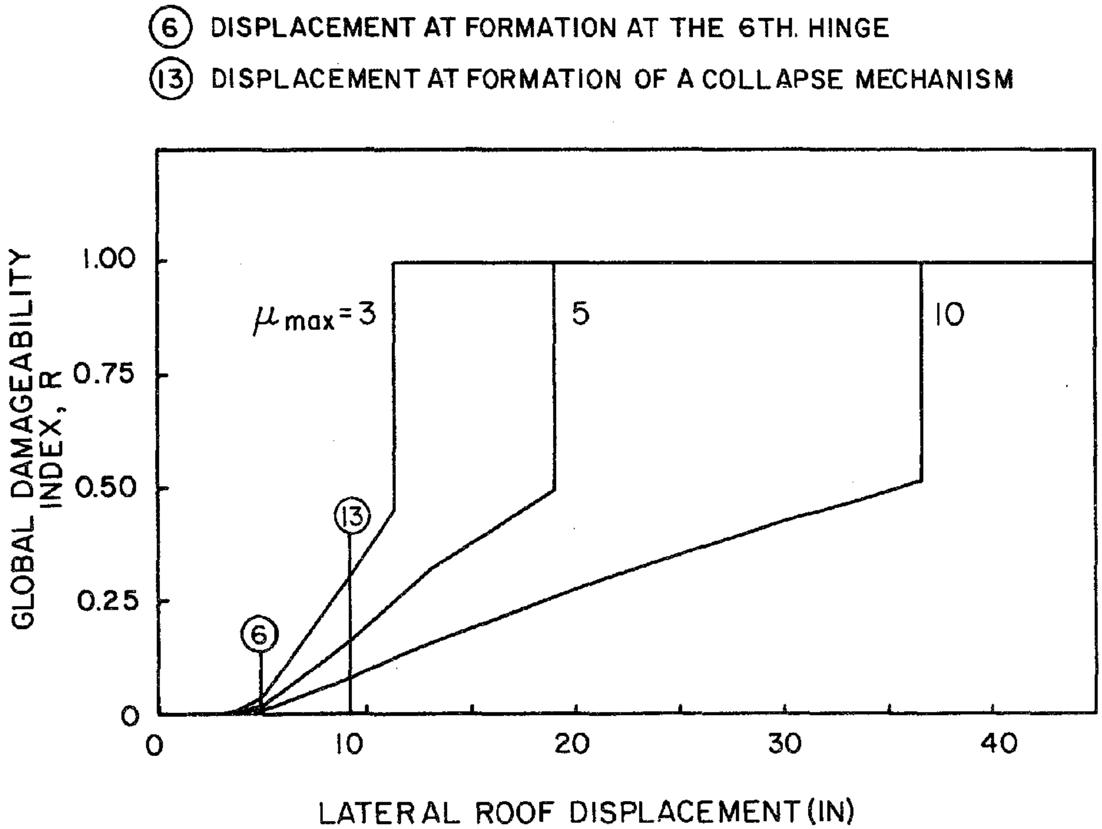


FIGURE 7.17 GLOBAL DAMAGEABILITY INDEX AND LATERAL ROOF DISPLACEMENT

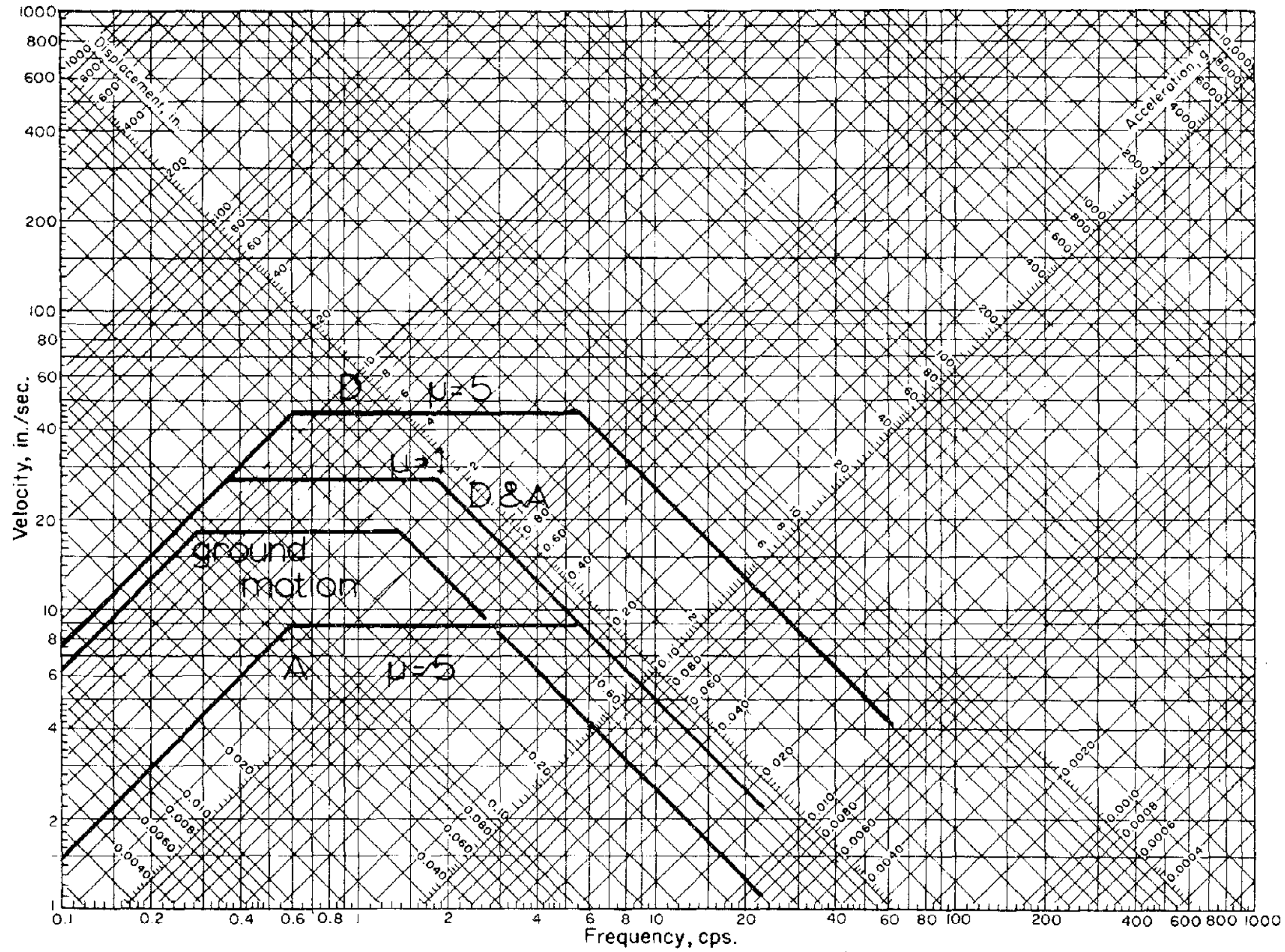


FIGURE 7.18 ELASTO-PLASTIC RESPONSE SPECTRA [15]

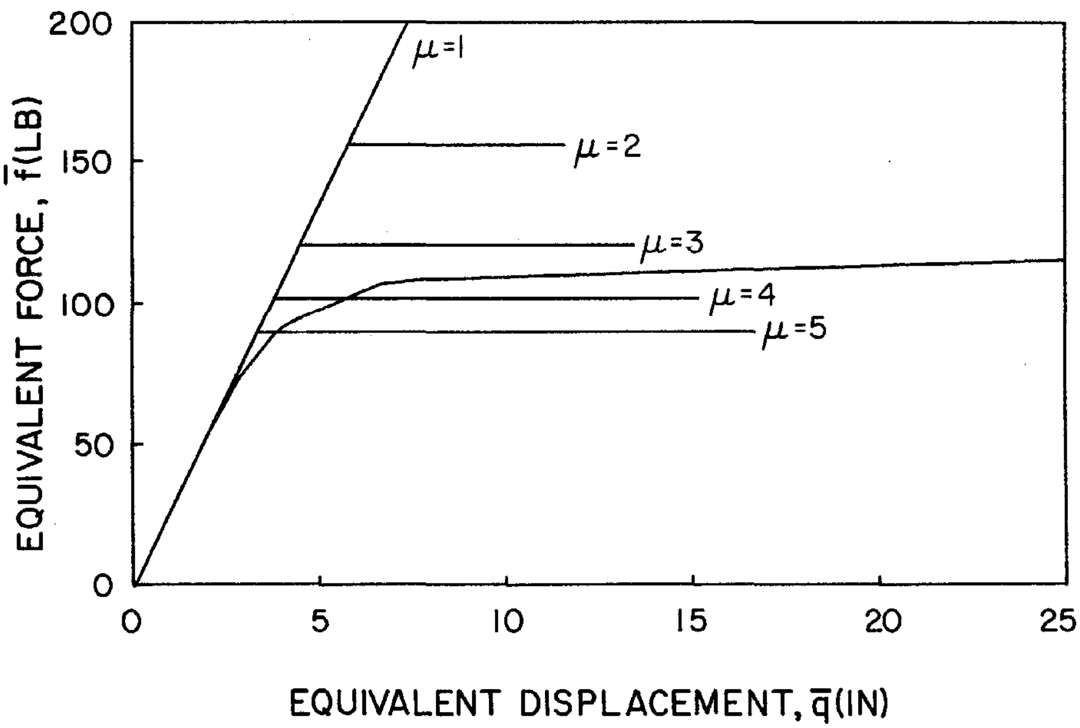


FIGURE 7.19 COMPARISON OF FORCE-DISPLACEMENT RELATIONSHIP FOR SPECTRA 1

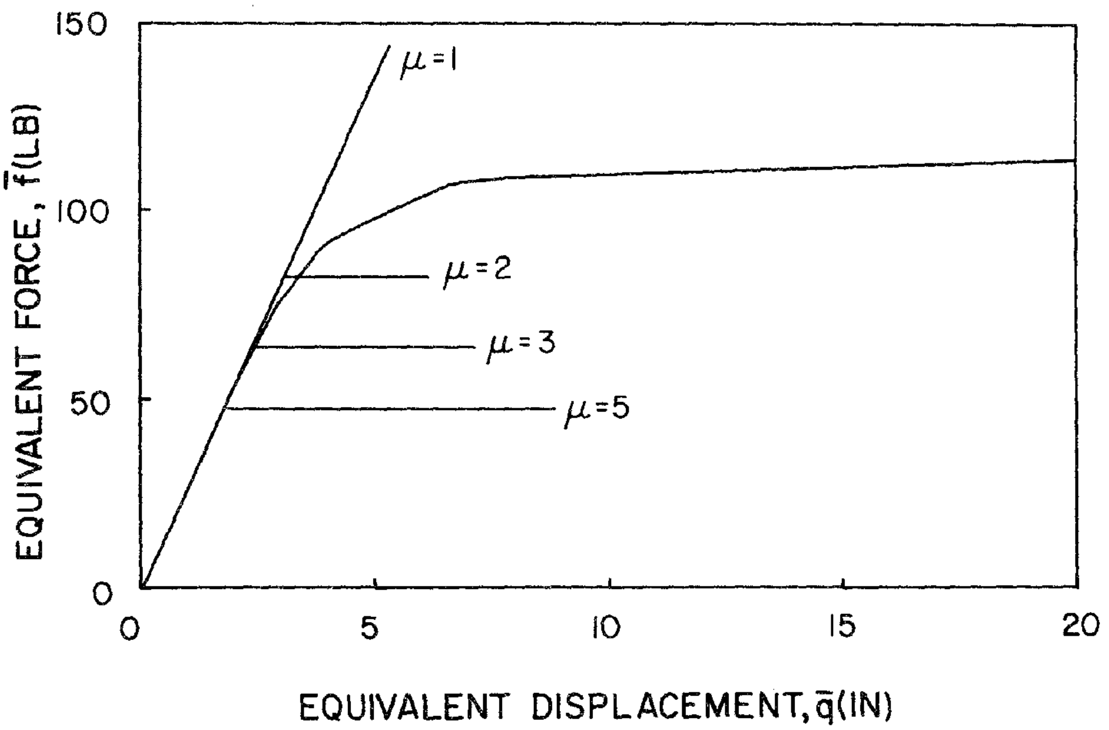


FIGURE 7.20 COMPARISON OF FORCE-DISPLACEMENT RELATIONSHIP FOR SPECTRA 2

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