EARTHQUAKE RESISTANT DESIGN AND ATC PROVISIONS

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SYNOPSIS

In general the object of earthquake resistant design is to enable a structure to resist with slight or no damage the earthquake motions that might reasonably be expected to occur during the lifetime of the structure, thus avoiding expensive repairs if a minor earthquake should occur. However, a more important purpose is to provide a large measure of resistance to, or to prevent, collapse or failure that might cause major property damage or loss of life, even in the event of a major earthquake or rare probability of occurrence. Although, for the first case, a structure may be designed to remain elastic or nearly so, in the latter instance it is unreasonably uneconomic to design for elastic behavior unless the structure is of such a character and of such importance that it might not be able to fulfill its intended use, even with slight damage, after the earthquake.

The seismic design procedures discussed herein are restricted essentially to buildings although some of the aspects of the topics discussed may be applied to other structures. The methods of analysis described are of two types: (1) moderately rigorous procedures of analysis to determine the response of structures essentially in the elastic range; and (2) approximate procedures suitable for use in design. Only a brief summary is given of the more rigorous procedures and attention is focused primarily on approximate procedures for design purposes. These procedures are embodied in the "Tentative Provisions for the Development of Seismic Regulations for Buildings," prepared by the Applied Technology Council (ATC). A brief summary of these provisions is contained in the paper.

RESUME

Les méthodes de calcul antisismique discutées ici se limitent essentiellement au bâtiment bien que certains aspects des sujets discutés peuvent s'appliquer à d'autres structures. Les méthodes d'analyse décrite appartiennent à deux types: (1) des méthodes modérément rigoureuses d'analyse pour déterminer la réponse des structures essentiellement dans la gamme élastique; et (2) des méthodes approximatives adéquates dont on peut se servir pour le calcul. On donne seulement un bref sommaire des méthodes plus rigoureuses et on insiste surtout sur des méthodes approximatives de calcul. Ces méthodes sont comprises dans le "Tentative Provisions for the Development of Seismic Regulations for Buildings," préparé par le "Applied Technology Council" (ATC). Un bref sommaire de ces dispositions est rapporté dans cette communication.

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INTRODUCTION

When a structure or a piece of equipment or instrumentation is subjected to earthquake motions, its base or support tends to move with the ground on which it is supported or with the element on which it rests. Since this motion is relatively rapid, it causes stresses and deformations in the item considered. If this component is rigid, it moves with the motion of its base, and the dynamic forces acting on it are very nearly equal to those associated with the base accelerations. However, if the component is quite flexible, large relative motions or strains can be induced in the component because of the differential motions between the masses of the component and its base. In order to survive the dynamic motions, the element must be strong enough as well as ductile enough to resist the forces and deformations imposed on it. The required strength and ductility are functions of stiffness or flexibility, among other things. In assessing seismic effects it should be remembered that the seismic actions generally are in addition to those already existing, i.e., arising from dead load, live load, thermal effects, etc.

Unfortunately, the earthquake hazard for which an element or component should be designed is subject to a high degree of uncertainty. In only a few areas of the world are there relatively long periods of observations of strong earthquake motions. The effects on a structure, component, or element, depend not only on the earthquake motion to which it is subjected, but on the properties of the element itself. Among these properties, the most important are the energy absorption within it or at interfaces between the element and its support, either due to damping or inelastic behavior, its period of vibration, and its strength or resistance.

EARTHQUAKE GROUND MOTIONS

The process of earthquake resistant design requires selection of the earthquake hazard as well as an estimate of structural strengths, either implicitly or explicitly, as an integral part of the procedure. Unless these determinations are made in a consistent manner, the final design may be either grossly uneconomical or dangerously unsafe. Both sets of parameters are probabilistic in nature although, for convenience, many of the aspects of the determination of structural strength may reasonably be approximated as deterministic. However, the earthquake motions themselves for which the design review is to be accomplished, or even the occurrence itself of an earthquake affecting the site, must be considered as probabilistic.

Regional Motions

In general, two procedures are available to define the earthquake hazard. In the first, where there is an extensive history of earthquake activity and geologic and tectonic investigations are feasible, estimates can be made of the possible magnitude and the location of future earthquakes affecting a site. In many instances, such earthquakes will occur along well defined faults. One can then make estimates of the earthquake motion intensity propagated to the site, taking into account the experimental and observational data available.

Donovan (Ref. 1) plotted data at various distances for accelerations from 678 world earthquake records ranging in magnitude from less than 5 to greater than 8. He found a great deal of scatter, which he was able to reduce somewhat by normalizing the data to the exponential of one-half the magnitude. He was able to show also that the probability distribution of the data is lognormal. For the median of the acceleration, a, measured in gravity units, g, Donovan derived a relation involving the hyperfocal slant range R, in km, measured from the earthquake focus to the point on the ground surface where the record was taken in terms of the Richter magnitude M, as given by the following equation:

$$a = 1.10\ e^{0.5M}\ (R + 25)^{-1.32} \qquad (1)$$

The geometric standard deviation, σ, the ratio of the median plus one standard deviation value to the median value, was very nearly 2.0, indicating that the spread in the data was quite large.

For data from 214 San Fernando records, Donovan obtained a larger attenuation and a smaller spread in the data, corresponding to the relationship (applying to the magnitude for this earthquake of 6.4):

$$a = 21.5\ g\ e^{6.4/2}\ (R + 25)^{-2.04} \qquad (2)$$

where the geometric standard deviation was determined to be 1.6. This more rapid attenuation has been noted by others, and is consistent with the data reported in Ref. 2.

In all cases, references to the figures will show that only very limited data exist for earthquakes closer than about 20 km to the hypocenter. The only definitive study of close-in earthquake motion is that contained in Ref. 3.

The second procedure for developing the earthquake hazard in a region is used when occurrence of earthquake is not generally associated with surface faulting, or when insufficient data are available from records and observations. Under these conditions, relationships have been developed for correlating ground motions, generally maximum velocities or maximum accelerations, to a qualitative measure of the intensity of motion, as for example that of the Modified Mercalli (MM) Intensity. Although these relations are not as readily subject to mathematical determination as the relations for earthquake shock propagation, there are sufficient observations to permit useful probabilistic data to be obtained. Such data are summarized in Refs. 4 and 5.

These data show even more scatter than those from accelerations and distance from the focus. They are complicated by the fact that the MM Intensity is a subjective measure in large part, and for higher levels of damage it depends to a great extent on the type of building, properties of building materials, foundation conditions and the like; for these reasons, for example,one would expect some changes in damage assessment over scores of years as the quality of construction materials improved. Data from quarry blasting indicates that plaster cracking rarely begins at less than 0.5 in/sec maximum ground velocity and generally is quite prevalent for velocities greater than 2 in/sec. Finally, the observation is made that in the El Centro earthquake of 1940, the maximum ground velocity was about 14 in/sec, and the Modified Mercalli Intensity was reported as IX.

These and other data suggest that the median value of the maximum ground velocity can be inferred from the Modified Mercalli Intensity by using the relationship that the maximum ground velocity is approximately 8 in/sec for MM VIII and changes by a factor of 2 for each unit change in MM Intensity below MM VIII, but increases above this level more slowly. It is believed that this relationship correlates well with observations from all dynamic sources. By comparison of the acceleration and velocity with the relationship that a velocity of 48 in/sec corresponds to a 1 g maximum acceleration in competent soils, one obtains the result that for Modified Mercalli Intensity VIII, the acceleration is 0.167 g and changes by a factor of 2 with each unit drop in MM Intensity. These relationships should drop off somewhat from the factor of 2 increase as the intensity increases above VIII, however.

It is believed that the relationship between maximum ground velocity and MM Intensity is nearly independent of the properties of the soil, but the relationship between velocity and acceleration is soil dependent and there may be some dependence of soil properties on the relationship for acceleration stated above. Nevertheless, the observations of MM Intensity are most strongly influenced by building type rather than by soil properties when intensity is associated with building damage. In other words, the soil type has implicitly been taken into account in the observation of damage or in the observational data leading to the MM Intensity reported.

Site Amplification and Modification

The regional motions that one derives from the methods described in the above must be modified to take account of the geologic and stratigraphic conditions pertaining to the site. Although there has been a great deal of study and research involved in this topic it must be considered still a controversial matter. Nevertheless, it is clear from observations that the type of soil or subsoil has a major influence on the motions that are recorded. In general, for the same earthquake, where the intensity is low (possibly maximum acceleration less than 0.2 g, where g is the acceleration of gravity) the measured accelerations are generally higher on sediments than on rock. However, when the acceleration is high (greater than 0.2 g), then the accelerations measured on rock appear to be higher than those on soil. In most instances the measured velocities are nearly the same. Studies of the nature of the motions on sites of different stiffnesses are summarized in Refs. 6 and 7 in terms of the so-called "response spectra" applicable to the measured records at various sites.

Although analytical methods have been proposed purporting to explain phenomena such as those described in the references previously cited, in most cases these analyses consider a condition not representative of actual conditions. The principal assumption (that the earthquake motions consist of horizontal shear waves propagated vertically upward from some base layer where the motions are defined) is contrary to observations.

For example, it is shown in Ref. 8, and it has long been considered, that for longer period motions, possibly where the periods are one second or longer, the motions are primarily due to surface waves such as Rayleigh waves or Love waves. It is quite likely, however, that for moderate distances, beyond those corresponding to the depth of focus, surface waves have an important effect even for higher frequencies or shorter period motions, and more complex motions must be considered other than those due to horizontal shears propagated vertically upward. Moreover, the fact that vertical motions occur cannot be accounted for by the simple horizontal shear wave model.

Considerations leading to variation in intensity of motion with depth beneath the surface are very complex. There are few data that directly relate surface motions to motions beneath the surface. The observational data for motions beneath the surface, compared with surface motions, includes two or three small earthquakes in Japan. These and other limited data indicate some reduction with depth of surface motion intensity, but for large motions or high intensities, they do not support the contention that one can compute accurately variations in intensities of motion with depth by methods involving only the vertical propagation of a horizontal shear wave.

It is not entirely rational to depend only on calculational methods to modify earthquake motions from some deep layer or bedrock to the surface. It would seem desirable to base inferences about site intensity modification on actual observations of surface motions as well as on calculations until such a time as measurements of motion become available from actual earthquakes at various depths beneath the surface for a number of different foundation conditions.

Several recent statistical studies have been made of vertical and horizontal earthquake motions (Refs. 9 and 10). Although the scatter in results is quite great, it has been recommended that the design motions in the vertical direction be taken as 2/3 of the value in the horizontal direction across the entire frequency range.

Actual Versus Effective Earthquake Motions

Although peak values of ground motion may be assigned to the various magnitudes of earthquake, especially in the vicinity of the surface expression of a fault or at the epicenter, these motions are in general considerably greater than smaller motions which occur many more times in an earthquake. Design earthquake response spectra are based on "effective" values of the acceleration, velocity and displacement, which occur several times during the earthquake, rather than isolated peak values of instrumental reading. The effective earthquake hazards selected for determining design spectra may be as little as one-half the expected isolated peak instrument readings for near earthquakes, ranging up to the latter values for distant earthquakes.

Design response spectra determined from these parameters can take into account the various energy absorption mechanisms, both in the ground and in the element, including radiation of energy into the ground from the responding system.

In the design of any system to resist seismic excitation, as discussed earlier herein, there are a number of parameters and design considerations that must be taken into account. Among these are the magnitude of the earthquake for which the design is to be made, the distance of the facility from the focus or fault, the parameters governing attenuation of motions with distance from the focus or epicenter, the soil or rock conditions as well as the general geologic conditions in the vicinity, and the parameters governing the response of the facility or the structure itself. Most, if not all, of these parameters are subject to considerable uncertainty in their value. Because so many of the parameters involved have probabilistic (rather than deterministic) distributions, it is not proper to take each of them with a high degree of conservatism because the resulting combined degree of conservatism would then be unreasonable. At the same time it is desirable to have an assured margin of safety in the combined design conditions. Hence, a choice must be made as to the parameters which will be taken with large margins of safety and those which will be taken with more reasonable values closer to the mean or expected values of the parameters.

The relation between magnitude of energy release in an earthquake and the maximum ground motion is very complex. There are some reasons for inferring that the maximum acclerations are, for example, nearly the same for all magnitudes of relatively shallow earthquakes for points near the focus or epicenter. However, for larger magnitudes, the values do not drop off so rapidly with distance from the epicenter, and the duration of shaking is longer. Consequently, the statistical mean or expected values of ground motions show a relationship increasing with magnitude, although not in a linear manner.

DYNAMIC STRUCTURAL RESPONSE

Dynamic Analysis

From the most general point of view, analysis of structural response to earthquake motion should consider as many components of ground motion as the number of degrees of freedom of the base: six for rigid base, many more for flexible foundations (spread footings without foundation girders), dams, bridges, etc. However, standard strong motion instruments do not record rotational components of ground motion and, in the past have usually not been installed in a dense enough grid to provide information on variation of ground motion over distances of the order of base dimensions of structures. In order to study their possible effects on structural response, ground rotations and spatial variations in translational motions have been estimated by idealized assumptions about the ground motion. In most analyses, however, no more than the three translational components (two horizontal and one vertical) of ground motion are considered.

Existing computer programs for inelastic, three-dimensional dynamic analysis of structures do not have an extensive collection of structural elements appropriate for buildings and are therefore of limited application in building analysis. Complete three-dimensional analysis of buildings is a formidable task. Usually separate planar models in the orthogonal horizontal directions are analyzed for one component of ground motion; computer programs have been developed to implement such analyses. However, planar analyses may not always be reasonable even for a building with nearly coincident centers of mass and resistance, because strong coupling between lateral motions in two orthogonal directions and torsional motions can arise if the lower natural frequencies are nearly coincident or due to asymmetric inelastic effects. In a model which has been employed to account for these coupling effects with a minimum of computational effort, the building is idealized as an assemblage of plane frames linked by rigid floor diagrams, with no enforcement of compatibility for vertical and rotational displacements at joints common to two or more frames. This model may be employed for buildings with floor diaphragms sufficiently stiff in their own plane relative to the vertical elements of the lateral force resisting structural system, and where the axial shortening of columns and lack of compatibility at joints common to two or more frames is not a significant factor in the response.

Results of nonlinear response history analysis of mathematical models of complex buildings would be reliable only if the model is representative of the building vibrating at large amplitudes of motion -- large enough to cause significant yielding. Extensive experimental research on force-deformation behavior of structural components at large deformations and shaking table studies on models of complete structures have improved our understanding of inelastic properties of structures. However, it is still difficult to construct mathematical models that lead to satisfactory results and are not complicated to the point of becoming impractical for analysis of complex structures. Furthermore, because dispersion in nonlinear responses to several design ground motions is rather large (Refs. 11 and 12), reliable results can be achieved only by calculating response to several representative ground motions -- recorded accelerograms and simulated motions -- and examining the statistics of response. Inelastic response history analyses, especially three-dimensional analyses, would therefore be an impractical requirement in the design of most buildings.

The most commonly used of the simpler methods of analysis are based on the approximation that effects of yielding can be accounted for by linear analysis of the building using the design spectrum for inelastic systems, determined from the elastic design spectrum and allowable ductility factor. (See, for example, Refs. 11, 13, 14, 15, 16) Forces and displacements due to each horizontal component of ground motion are separately determined by analysis of an idealization of the building with one lateral degree of freedom per floor in the direction of the ground motion being considered. Such analysis may be carried out by either the modal analysis procedure or a simpler method which will be referred to as the equivalent lateral force procedure. Both procedures lead directly to lateral forces in the direction of the ground motion component being considered. The main difference between the two procedures lies in the magnitude and distribution of the lateral forces over the height of the building. In the modal analysis procedure the lateral forces are based on properties of the natural vibration modes of the building, which are determined from the distribution of mass and stiffness over height; but in the equivalent lateral force procedure the magnitude of forces is based on an estimate of the fundamental period, and their distribution on simple formulas appropriate for regular buildings. Otherwise the two procedures have similar capabilities and are subject to the same limitations.

The direct results from either procedure are for the effects of lateral forces in the direction under consideration: story shears, floor deflections, and story drifts. The story moments also are obtained directly in the modal analysis procedure but a correction factor has to be introduced in the equivalent lateral force procedure. The following effects are all considered in the same simple, indirect manner in the two procedures: effects of the horizontal component of ground motion perpendicular to the direction under consideration in the analysis, of torsional motions of the structure, of vertical motions of the structure due to horizontal ground motions, of the vertical component of ground motion, and of gravity loads -- the so-called P-delta effects.

A preliminary design of the building must be available before the modal analysis procedure or any of the more rigorous procedures can be implemented, because these procedures require the mass and stiffness properties of the building. The equivalent lateral force procedure is ideally suited for preliminary design, and is needed even if more refined analysis procedures will be used in the final design process.

Design Response Spectra

The response spectrum (Refs. 11, 16) is defined as a graphical relationship of maximum response of a single-degree-of-freedom elastic system with damping to dynamic motion or forces. The most usual measures of response are maximum displacement, D, which is a measure of the strain in the spring element of the system, maximum pseudo relative velocity, V, which is a measure of the energy absorption in the spring of the system, and maximum pseudo acceleration, A, which is a measure of the maximum force in the spring of the system. Although actual response spectra for earthquake motions are quite irregular, they have the general shape of a trapezoid or tent: a simplified spectrum is shown in Fig. 1, plotted on a logarithmic tripartite graph, and modified so that the various regions of the spectrum are smoothed to straight line portions. On the same graph are shown the maximum ground components, and the figure therefore indicates the amplifications of maximum ground motions for the various parts of the spectrum. Values of amplification factors for various amounts of damping are shown in Table 1 for two levels of probability considering the variation as lognormal.

At any specific frequency, f, the relations between the values of D_f, V_f, and A_f are defined as follows:

$$V_f = \omega D_f \tag{3}$$

$$A_f = \omega V_f = \omega^2 D_f \tag{4}$$

where ω is the circular natural frequency, $2\pi f$.

Recommended damping values for various materials and structural types are given in Table 2.

Let us consider the case in which a simple mass-spring oscillator deforms inelastically with a relation of resistance to relative displacement as in Fig. 2. It is convenient to use an elasto-plastic resistance displacement relation because one can draw response spectra for such a relation in generally the same way as spectra are drawn for elastic conditions.

In Fig. 2, the elasto-plastic approximation to the actual curvilinear resistance-displacement curve is drawn so that the areas under both curves are the same at the "effective" elastic displacement u_y and at the selected value of maximum permissible displacement u_m. The "ductility" factor μ is defined as

$$\mu = u_m/u_y \tag{5}$$

In Fig. 3, there are shown both elastic and inelastic spectra for both acceleration and total displacement. Here the symbols D, V, A, A_o refer to the bounds of the elastic spectrum, the symbols D', V', A', A_o to the bounds of the elasto-plastic spectrum for acceleration, and the symbols D, V, A'', A_o'' to the bounds for the elasto-plastic spectrum for displacement. The symbol A_o refers to the maximum ground acceleration.

In general, for small excursions into the inelastic range, when the latter is considered to be approximated by an elasto-plastic resistance curve, the acceleration response spectrum is decreased by a factor which is one over the ductility factor, $1/\mu$. Then the reduction for the two left-hand portions (D and V) of the elastic response spectrum shown in Fig. 3 (to the left of a frequency of about 2 hertz) is by the factor $1/\mu$, and by the factor of $1/\sqrt{2\mu - 1}$ in the constant acceleration portion (A) to the right, roughly between frequencies of 2 and 8 hertz. There is no reduction beyond about 33 hertz. With this concept, one can arrive at design spectra that take account of inelastic action even in the small range of inelastic behavior. The method of drawing Fig. 3 is illustrated in more detail by Fig. 4.

Modification of Spectra for Large Periods or Very Low Frequencies

The response spectra of Figs. 1 and 3 have a constant velocity response, V, in the range of frequencies below about 2 hertz, with a cutoff to a constant displacement line below about 0.2 hertz. For structures with long periods, greater than about 1 sec, the spectral values are not conservative enough for the lower modes of vibration in the modal analysis procedure and definitely not conservative enough for the equivalent lateral force procedure. In order to arrive at a more conservative spectrum for the design, taking account of various uncertainties involved in the combination of modal responses, and other factors, the following procedure is suggested. In the range of frequencies below 1 hertz, the velocity spectral response value should be taken to vary as the reciprocal of the frequency to the 1/3 power, or as the period to the 1/3 power, instead of being constant as shown in the figures. This corresponds to spectral acceleration values in this range varying as the reciprocal of the period to the 2/3 power, or directly as the frequency to the 2/3 power, instead of varying as the reciprocal of the period or directly as the frequency, as in the figures. In either case, it is suggested that the spectrum be considered to correspond to a constant displacement equal to the amplified ground displacement for periods longer than about 6 sec, or frequencies lower than about 1/6 hertz.

Building Properties and Allowable Ductility Factor for Analysis

Mass and stiffness -- The masses to be used in the analysis of a building include all of the fixed dead and live loads, plus reasonably probable values of the variable or movable live loads. In general it need not be assumed that the design live load for which the floor is designed, for example, should be considered as applied everywhere on each floor at the same time in determining the masses at the various levels in a building subjected to earthquake motions. It is appropriate to use the same proportion of live load that is used as a cumulative factor in the design of the columns in the building. Snow loads can be considered as live loads in the same way, with some average snow load considered as a mass that must be taken into account in the design of the building frame.

In the computation of stiffness of steel members and framing, the behavior of the joints and connections must be considered. This is best taken into account by using center-to-center distances of members between joints, with consideration of the increased stiffness properties near the joints of welded construction, or possibly a decrease for riveted or bolted construction in which the bolts are not high tensile bolts. In reinforced concrete sections, the members are not likely to be completely uncracked, nor is it reasonable to assume that all sections have the reduced stiffness that a cracked section has. For this reason, it is appropriate to use an average of the moments of inertia for stiffnesses between cracked and completely uncracked sections or as between net and gross sections for reinforced concrete members, unless they clearly are stressed at such low levels that cracking is not likely. Interaction of floor systems with other transverse members such as beams and girders should be considered where the floor system acts as a stiffening element in flexure.

The contribution of nonstructural elements both to mass and to stiffness should be considered in the design even though these may not be used in developing the required strength of the structure. The added stiffness and mass may contribute to greater moments which must be resisted by the load bearing elements of the structure.

Damping and ductility -- Damping levels are of course dependent on the level of deformation or strain in a structure. This is reflected in the recommended damping values given in Table 2, where values are suggested in which the percentages of critical damping are given for working stress levels or stress levels no more than 1/2 the yield point, and for levels of deformation corresponding to stresses at or just below yield levels. In the table the lower values are to be used for structures in which considerable conservatism in the design is desirable, and the upper levels for ordinary structures in general.

Ductility levels for structures are used in a way which involves a general reduction in the design spectrum. Hence some reasonable assessment of the allowable ductility factor is required. For this purpose one must be aware of the differences between the various kinds of ductilities involved in the response of structures to dynamic loading. In this respect one must make a distinction between the ductility factor of a member, such as the rotational hinge capacity at a joint in a flexural member, the ductility factor of a floor or story in a building, and the overall ductility factor of the building for use in the computation of base shear from the response spectral values.

The ductility factor of a member, or of a floor, or the overall ductility factor are all governed by the development of a resistance-displacement relation, with the displacement being the longitudinal deformation in a tensile or compression member, the rotation at a joint or connection in a flexural member, or the total shearing deformation in a shear wall. The story ductility factor is essentially defined by use of a relationship in which the displacement is the relative story deflection between the floor above and the floor beneath. The overall ductility factor is some weighted average in general of the story ductility factors, and is defined best by considering a particular pattern of displacement corresponding to the preferred mode of deformation of the structure in a response condition in which inelastic energy is absorbed as generally as it is possible to develop such deformation throughout the structure.

It can be seen from the discussion that the member ductility factor may be considerably higher than the story ductility factor, which in turn may be somewhat higher than the overall ductility factor. In order to develop an overall ductility factor of 3 to 5 in a structure, the story ductility factors may have to vary between 3 to 8 or 10, and the individual member ductility factors probably will lie in the range of 5 to 15 or even more. In this regard it must be remembered that the ductility factor as defined herein is given by the ratio of the maximum permissible deformation to the deformation at the "effective" yield displacement, rather than the ratio of the maximum deflection to the elastic limit deflection or displacement, as shown in Fig. 2.

Ductility factors for steel are generally higher than for reinforced concrete, and for steel structures ductility factors are higher for tension than for bending and higher for bending than for compression. Ductility factors in shear are intermediate between the values in bending and compression, generally. However, the development of high ductilities in flexure or in compression requires that the thickness of outstanding unsupported legs of members be limited in general to a value of the order of a thickness greater than 1/6 the width of the outstanding leg to develop a ductility factor of the order of 6 or so in compression or on the compression side of flexural members. In any case, the yield value for a member that buckles either overall or locally is at some level just below the load where buckling or wrinkling begins, and is generally quite limited.

For reinforced concrete, the ductility factor is a function of the state of stress and the arrangement of reinforcement. Ductility factors of the order of 10 or more in flexure of reinforced concrete beams with equal amounts of compression and tension reinforcement are not difficult to attain. However, without the compressive reinforcement, the ductility factor is less for higher percentages of steel and is inversely proportional to the amount of steel, with a value of the order of 10 being the maximum for 1 percent of tensile steel reinforcement. In members in shear the ductility factor reached is also a function of the arrangement and placement of steel, and in general does not exceed values of the order of about 3, possibly even less for members with a high amount of compression such as reinforced concrete columns. However, higher ductility factors can be reached if the concrete subjected to high compression is contained in some manner, such as by spiral reinforcement, in which case ductility factors of the order 4 to 6 may be reached. For shear walls with diagonal as well as horizontal and vertical reinforcement, ductility factors of 4 to 6 are possible. In timber, ductility factors of the order of 2 to 4 are possible, and in masonry lower ductility factors, of the order of 1 to 3, may be reached depending upon whether the masonry is reinforced or not. A maximum of about 1.3 is the upper limit for unreinforced masonry.

In a building, the story ductility factor should not vary rapidly nor should there be a major change in the rate of increase or decrease in story ductility factor with height. Preferably the exercise of control on the story design capacity should be based on the relationship of the shear strength permitted in that story, relative to the computed story shear. In general for use of a ductility factor in the range of 5 to 6, the smallest ratio of story shear capacity to computed story shear should not be less than 80 percent of the average of these ratios for all stories. For a story ductility of 4 the corresponding ratio need be only 0.67.

Modal Analysis Procedure

The modal method or the mode superposition method (Refs. 11, 12) is generally applicable to analysis of dynamic response of complex structures in their linear range of behavior, in particular to analysis of forces and deformations in multistory buildings due to medium intensity ground shaking causing moderately large but essentially linear response of the structure. The method is based on the fact that, for certain forms of damping -- which are reasonable models for many buildings -- the response in each natural mode of vibration can be computed independently of the others, and the modal responses can be combined to determine the total response. Each mode responds with its own particular pattern of deformation, the mode shape, with its own frequency, the modal frequency, and with its own modal damping; and the modal response can be computed by analysis of a single-degree-of-freedom (SDOF) oscillator with properties chosen to be representative of the particular mode and the degree to which it is excited by the earthquake motion. Independent SDOF analysis of the response in each natural vibration mode is, of course, a very attractive feature of modal analysis. Even more significant is the fact that, in general, the response need be determined only in the first few modes because response to earthquakes is primarily due to the lower modes of vibration. For buildings, numerous full-scale tests and analyses of recorded motion during earthquakes have shown that the use of modal analysis with viscously damped single-degree-of-freedom oscillators describing the response of vibration modes, is an accurate approximation for analysis of linear response.

A complete modal analysis provides the history of response -- forces, displacements and deformations -- of a structure to a specified ground acceleration history. However, the complete response history is rarely needed for design; the maximum values of response over the duration of the earthquake usually suffice. Because the response in each vibration mode can be modeled by the response of a SDOF oscillator, the maximum response in the mode can be directly computed from the earthquake response spectrum, and procedures for combining the modal maxima to obtain estimates (but not the exact value) of the maximum of total response are available.

Strictly speaking, the modal method, which is applicable only to analysis of linear response, cannot be used for calculation of the design forces for buildings, because they are usually designed to deform significantly beyond the yield limit during moderate to very intense ground shaking. However, it is believed that for many buildings satisfactory approximations to the design forces and deformations can be obtained from the modal method by using the design spectrum for inelastic systems instead of the elastic response spectrum. In what follows, the modal method is presented first for elastic systems and later for inelastic systems; the presentation is based on recent design recommendations (Ref. 17).

In its most general form, the modal method for linear response analysis is applicable to arbitrary three-dimensional structural systems. However, for purposes of design of buildings it can often be simplified from the general case by restricting consideration to lateral motion in a plane. Planar models appropriate for each of two orthogonal lateral directions are analyzed separately and results of the two analyses are then combined.

Structural idealization -- The mass of the structure is lumped at the floor levels; only one degree of freedom -- the lateral displacement in the direction for which the structure is being analyzed -- per floor is required, resulting in as many degrees of freedoms as the number of floors.

Modal periods and shapes -- The periods and shapes of vibration are required for each of those natural modes of vibration which may contribute significantly to the total design quantities. These should be associated with moderately large, but essentially linear response of the structure, and the calculations should include only those building components which are effective at these amplitudes. Such periods may be longer compared to those obtained from a small-amplitude test of the building when completed, or the response to small earthquake motions, because of the stiffening effects of the non-structural and architectural components at small amplitudes. During response to strong ground shaking, however, the measured responses of buildings have shown that the periods lengthen, indicating the loss of stiffness contributed by these components.

Several methods for calculating natural periods and associated mode shapes of vibration of a structure are available (Refs. 11, 12). These calculations can be carried out readily by standard computer programs that are widely available.

As mentioned earlier, responses of buildings to earthquake motion are usually due mostly to the first few modes of vibration. For determining design values of forces and deformations, three modes of vibration in each lateral direction are nearly always sufficient for low- and medium-rise buildings, but more modes may be necessary in the case of high-rise buildings; six modes in each direction would generally be sufficient.

Modal responses -- Maximum responses in each natural mode of vibration can be expressed in terms of the modal properties and the earthquake response spectrum. For the nth mode, the base shear component α_{on} is:

$$\alpha_{on} = \frac{A_n}{g} \beta_n \tag{6}$$

in which A_n is the ordinate, corresponding to the nth mode of vibration having the natural period T_n and damping ratio ξ_n, of the pseudo-acceleration response spectrum; g is the acceleration of gravity; and β_n the effective weight or the portion of the weight of the building that participates in the nth mode.

$$\beta_n = \frac{\left[\sum_{i=1}^{N} w_i\ \varphi_{in}\right]^2}{\sum_{i=1}^{N} w_i\ \varphi_{in}^2} \tag{7}$$

where w_i is the mass lumped at the ith floor level, φ_{in} is the modal displacement of the ith floor, and N is the total number of floor levels. Eq. (7) will give values of β_n that are independent of how the modes are normalized. The lateral force at the ith floor level in the nth mode of vibration is

$$q_{in} = \alpha_{on} \frac{w_i\ \varphi_{in}}{\sum_{j=1}^{N} w_j\ \varphi_{jn}} \tag{8}$$

In applying the forces at the various floor levels to the building, their direction is controlled by the algebraic sign of φ_{in}. Hence, the modal forces for the fundamental mode will act in the same direction; for the second or higher modes they will change direction as one moves up the structure. The form of Eqs. (6) and (8) is different from that usually employed, and is chosen here to highlight the relationship between the modal analysis procedure and the equivalent lateral force procedure.

The subsequent calculation of internal forces -- story shears and story moments -- and deflections associated with the lateral forces for each mode does not involve any dynamic analysis. The lateral loads are applied at each floor level, statics is used to calculate the story shears and story moments, and a static deflection analysis to determine the floor deflections. The latter is not necessary, however, because deflections due to the lateral forces associated with a particular mode, Eq. (8), are proportional to the mode shape, and the two are related rather simply.

$$v_{in} = \frac{1}{\omega_n^2} \frac{g}{w_i} q_{in} \tag{9}$$

where $\omega_n = 2\pi/T_n$ is the frequency of the nth natural mode of vibration.

The story shears and moments in individual modes are combined to determine their total values. These total values are distributed to the various frames and walls that make up the lateral force resisting system. Whereas it is convenient in all cases and satisfactory for many buildings to defer such distribution until after the modal values of story forces have been combined, the results may be in error for walls and braced frames. For better evaluation of shears and moments at various levels in walls and braced frames, individual modal values for these quantities should be determined by appropriate distribution of the modal values of story shears and moments and combined directly.

Similarly deformation quantities should not be determined from the total (after combining the modal values) floor displacements, but individual modal values should be determined and combined. For example, Δ_i, the drift in story i should be determined by combining the modal values

$$\Delta_{in} = v_{in} - v_{i-1,n} \tag{10}$$

Total responses -- As mentioned earlier, total responses of an elastic structure are the superposition of responses in the natural modes of vibration of the structure, and the maximum responses in individual modes of vibration can be determined from the earthquake response spectrum. Because, in general, the modal maxima r_n do not occur simultaneously during the ground shaking, they cannot be directly superimposed to obtain r, the maximum of the total response. The direct superposition of modal maxima, however, provides an upper bound to the maximum of total response:

$$r \leq \sum_{n=1}^{N} |r_n| \tag{11}$$

This estimate is often too conservative and is therefore not recommended. A satisfactory estimate of the total response can usually be obtained from the root-sum-square:

$$r \simeq \sqrt{\Sigma r_n^2} \tag{12}$$

in which, as discussed earlier, only the lower few modes need to be included in the summation.

The maximum value of any response -- story shear, story moment, shear and moment at various levels in braced frames and walls, floor displacement, story drift, etc. -- can be estimated by combining the modal values for that response in accordance with Eq. (12). The quality of this estimate is generally good for systems with well separated frequencies, a property typically valid for the building idealization adopted here, wherein only the lateral motion in a plane is considered. Improved combination formulas are available for systems lacking this property (Ref. 11).

Application to inelastic systems -- The modal analysis procedure described above is strictly valid only for systems in their linearly elastic range of behavior. With the following modifications, it may, however, be employed as an approximate procedure for analysis of nonlinear responses. In Eq. (6), replace A_n, the ordinate of the pseudo acceleration response spectrum for a linearly elastic system with vibration period T_n and damping ratio ξ_n by A_n', the corresponding value for a nonlinear system, with the same period of small amplitude vibration and damping ratio, which is determined from the design response spectrum for the allowable ductility factor is as follows:

$$\alpha_{on} = \frac{A_n'}{g} \beta_n \tag{13}$$

Multiply displacements calculated from Eq. (9) by μ, the allowable ductility factor, to obtain the total deflection in the nth mode.

$$v_{in} = \mu \frac{1}{\omega_n^2} \frac{g}{w_i} q_{in} \tag{14}$$

Earthquake design responses -- Two independent analyses by the modal analysis procedure described above lead to the effects of lateral forces associated with ground motion in two orthogonal directions. The design forces and deformations due to earthquake effects are determined by combining the results of these independent analyses, and including the effects of torsional motions of the structure, of vertical motions of the structure due to horizontal ground motion, of the vertical component of ground motion, and the P-delta effects.

Equivalent Lateral Force Procedure

Although the building is idealized in the same manner as in the modal analysis procedure, the equivalent lateral force procedure requires less effort because periods and shapes of natural modes of vibration are not needed. The magnitude of lateral forces is based on an estimate of the fundamental period of vibration and their distribution on simple formulas appropriate for buildings with regular distribution of mass and stiffness over height. Situations where results from the equivalent lateral force procedure may not be satisfactory are discussed later. A number of versions of this procedure, differing in detail but based on the same underlying concepts, can be found in various building codes.

Planar models appropriate for each of two orthogonal lateral directions are analyzed separately as described next; the results of the two analyses are combined as discussed later.

Fundamental period of vibration -- Methods of mechanics cannot be employed to calculate the vibration period before a design, at least a preliminary one, of the building is available. Simple formulas which involve only a general description of the building type -- e.g., steel moment frame, concrete moment frame, shear wall system, braced frame, etc. -- and overall dimensions such as height and plan size -- are therefore necessary to estimate the vibration period so that the base shear can be computed to provide the initial design. Because pseudo-acceleration values in design spectra for inelastic systems with moderate to large values of allowable ductility factor generally decrease with increasing values of vibration period, it is desirable to underestimate the fundamental period so that the computed base shear is conservative.

A formula for moment-resisting frame buildings is recommended in the ATC Code (Ref. 17).

$$T = C_T \, H^{3/4} \tag{15}$$

where C_T = 0.035 and 0.025 for steel and concrete frames, respectively, and H is the height in feet of the building. A commonly used formula for reinforced concrete shear-wall buildings and braced steel frames is:

$$T = \frac{0.05\,H}{\sqrt{L}} \tag{16}$$

where L is the plan dimension in feet in the direction of analysis.

The fundamental vibration period of exceptionally stiff or light buildings may be significantly shorter than the estimate provided by formulas such as Eqs. (15) and (16). Especially for such buildings and as a general check, the period for an initial design of the building should be computed by established methods of mechanics. An approximate formula, based on Rayleigh's method, is especially convenient:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{N} w_i\, v_i^2}{g \sum_{i=1}^{N} q_i\, v_i}} \tag{17}$$

in which the v_i (i=1, 2,...N) quantities are the static lateral displacements at the various floor levels, computed on a linear elastic basis, due to a set of lateral forces q_i. Any reasonable distribution for q_i may be selected but it is convenient to use the lateral forces computed, as described later in this section from the empirical estimate of the fundamental period.

Nonstructural elements participate in the behavior of the structure even though the designer may not rely on them for contributing strength or stiffness to the structure. To ignore them would lead to longer periods and usually smaller design forces; hence they should be considered in calculating the period.

Lateral forces -- The distribution of lateral forces over the height of a building is generally quite complex because a number of natural modes of vibration contribute significantly to these forces. The contributions of the various vibration modes to the lateral forces and to the base shear depend on a number of factors, including shape of the design response spectrum, and natural vibration periods and mode shapes -- which in turn depend on the mass and stiffness properties of the building. However, these forces are, in a large part, due to the first (fundamental) mode of vibration. Thus, in the equivalent lateral force procedure, they are determined from formulas similar to those for the first mode, using Eqs. (6), (7), and (8), but modified to account approximately for the effects of the higher modes.

The following formulas for α_0, the base shear, and f_i, the lateral force at each floor i, have been recommended (Ref. 17):

$$\alpha_o = \frac{A_1'}{g} W \tag{18}$$

in which A_1' is the pseudo-acceleration corresponding to the estimated fundamental period and the appropriate damping, determined from the design response spectrum for allowable ductility factor;

$W = \sum_{i=1}^{N} w_i$ is the total weight of the building;

$$f_i = \alpha_o \frac{w_i h_i^k}{\sum_{j=1}^{N} w_j h_j^k} \qquad (19)$$

in which h_1 is the height of the ith floor above the base and k is a coefficient related to the estimated fundamental vibration period as follows:

$$k = \begin{cases} 1 & T \leq 0.5 \text{ sec} \\ (T + 1.5)/2 & 0.5 < T < 2.5 \text{ sec} \\ 2 & T \geq 2.5 \text{ sec} \end{cases} \qquad (20)$$

The basis for these formulas is presented in the following paragraph.

If β_1, the effective weight for the first mode, is replaced with W, the total weight, Eq. (13) with n = 1 will become identical with Eq. (18) provided the same value of the fundamental period is used to determine A_1' in both cases. β_1 will always be smaller than W; typically values for β_1 are between 60 to 80 percent of W, depending on the distribution of weight over the height and the shape of the first mode. Eq. (18) would therefore provide a value for the base shear which will be significantly larger than the first mode value; thus it indirectly and approximately accounts for the contributions of the higher modes of vibration.

If α_{o1} is replaced with the total base shear α_o , Eq. (8) will become identical with Eq. (19) with k = 1 if the first mode shape is a straight line; with k = 2 if the first mode shape is a parabola with vertex at the base. Eq. (19) with k = 1 is appropriate for buildings having fundamental period of 0.5 sec or less, because the influence of vibration modes higher than the fundamental mode is small in earthquake responses of short-period buildings, and the fundamental vibration mode of regular buildings departs little from a straight line. Although earthquake responses of long-period buildings are primarily due to the fundamental mode of vibration, the influence of higher modes can be significant, and the fundamental mode lies between a straight line and a parabola with vertex at the base. The force distribution of Eq. (19) with k = 2 is therefore appropriate for buildings having fundamental period of 2.5 sec or longer. Linear variation of k between values of 1 at a period of 0.5 sec, and 2 at 2.5 sec provides a simple transition between the two extreme values.

Story forces -- Story shears are related to the lateral forces by equations of statics. The shear in any story is simply the sum of the lateral forces at floor levels above that story.

The story moments can be similarly determined from the lateral forces and heights of various stories by methods of statics. However, there are reasons for reducing the statically computed overturning moments to obtain design values:

1. The distribution of story shears over height computed from the lateral forces of Eq. (19) is intended to provide an envelope; because of the contributions of several modes, shears in all stories do not attain their maximums simultaneously during an earthquake. Thus the story moments statically consistent with the envelope of story shears will be overestimates.

2. It is intended that the design shear envelope based on the simple distribution of lateral forces of Eq. (19) be conservative. If the shear in some story is close to the exact value, the shears in almost all other stories are likely to be overestimated. Hence, the story moments statically consistent with the design shears will be overestimates.

3. Under the action of overturning moments, one edge of the foundation may lift from the ground for short durations of time. Such behavior leads to substantial reduction in the axial forces in columns and other vertical members, caused by overturning.

Results of dynamic analysis taking into account the first two of the foregoing reasons suggest that up to 20 percent reduction is, in general, reasonable for the story moments computed statically from the envelope of story shears, but no reduction should be permitted in the upper stories of a building. Rotational inertia due to axial deformations of columns and/or base rotation, in turn caused by soil-structure interaction or rotational components of ground motion, increases moments considerably, near the top. In any case, there is hardly any benefit in reducing the story moments near the top of buildings, because design of vertical elements near the top is rarely governed by these moments. Consequently, the following values have been recommended for the reduction factor by which the statically computed story moments should be multiplied: 1.0 for the top 10 stories; between 1.0 and 0.8 for the next 10 stories from the top, linearly varying with height; and 0.8 for the remaining stories.

Formerly many building codes and design recommendations, including the 1968 SEAOC recommendations (Ref. 18), allowed large reduction in overturning moments relative to their values statically consistent with the story shears. These reductions appeared to be excessive in light of the damage to buildings during the 1967 Caracas earthquake, where a number of column failures were primarily due to the effects of overturning moment. In later (1973) versions of the SEAOC recommendations (Ref. 19), no reduction was allowed. However, making no reduction at all is too conservative in light of the reasons mentioned above and the results of dynamic analysis.

Methods for distributing story shears and moments to the various frames and walls that make up the lateral force resisting system are presented below.

Deflections and drifts -- A static deflection analysis that assumes linear behavior of the building will lead to a set of floor deflections. In order to account for the inelastic effects, these deflections should be multiplied by the allowable ductility factor that was used in establishing the design spectrum, resulting in the total deflections. The drift in a story is computed as the difference of the deflections of the floors at the top and bottom of the story under consideration.

Earthquake design responses -- Effects of lateral forces associated with ground motion in two orthogonal directions can be determined from two independent analyses by the equivalent lateral force procedure presented above. The design forces and deformations due to earthquake effects are determined by combining the results of these independent analyses, and including the other effects mentioned above.

SPECIAL DESIGN CONSIDERATIONS

Torsion

Torsional responses in structures arise from two sources: eccentricity in the mass and stiffness distributions in the structure, causing a torsional response coupled with translational response; and torsion arising from accidental causes, including uncertainties in the masses and stiffnesses, the differences in coupling of the structural foundation with the supporting earth or rock beneath; and wave propagation effects in the earthquake motions that give a torsional input to the ground as well as torsional motions in the earth itself during the earthquake.

In general, the torsion arising from eccentric distributions of mass and stiffness can be taken into account by determining the distance between the center of mass and the center of shear stiffness, and ascribing an incremental torsional moment in each story corresponding to the shear in that story multiplied by this eccentricity. A precise evaluation of the torsional response is quite complicated because it is necessary to make essentially a two- or three-dimensional response calculation, taking into account the coupled modes of response of the entire structure. However, one can approximate the response by summing from the top story the incremental torsional moments computed as described above, to obtain the total torsional moment in each story.

The "static" torsional responses in each story are then determined by computing the twist in each story obtained by dividing the total torsional story moment by the story rotational stiffness. These twists are then added from the base upward to obtain the total twisting or torsional response at each floor level.

Since these are static responses, they should be amplified for dynamic response using the response spectrum amplification factor. It is probably adequate to use the factor corresponding to the response spectrum amplification factors in Table 1 for the fundamental torsional frequency of the structure. However, in many design codes no amplification whatsoever is used.

Accidental torsion may arise in various ways. One can take a value of accidental eccentricity of the order of about 5 to 10 percent of the width of the structure in the direction of motion considered to account for the accidental torsional response. Most current building codes use a value of 5 percent. If one does this, one can consider the accidental torsion as an increase and also as decrease in the eccentricity corresponding to the distance between the centers of mass and resistances in the various stories, with consideration of increases in all levels or decreases in all levels to get two bounding values. The accidental torsion or the total torsion is computed in the same way as the "real" torsion described above.

Distribution of Shears

The story shears arising from translational and from torsional response are distributed over the height of the building in proportion to the stiffnesses of the various elements in the building, with the translational shears being affected by the translational stiffnesses of the building, and the torsional shears being affected by the rotational stiffnesses of the building. The computed stiffness of the structure should take into account the stiffness of the floors and floor structure acting as a diaphragm or distributing element. The floor diaphragm can be considered infinitely stiff, and only the story stiffnesses are of importance. However, if the floor diaphragm is flexible and deforms greatly, the distribution of the forces becomes more nearly uniform than determined by the method discussed above. A simplified approach is possible by considering the relative displacements of the building due to translation, and that due to rotation of each story separately, as affected by the diaphragm or floor stiffness, with the stiffnesses being determined by the forces corresponding to a unit displacement in either translation or torsion, respectively. Then one distributes the shears due to translation or rotation in proportion to these stiffnesses.

Base or "Overturning" Moments

The flexural moment about a horizontal axis at the base is of importance in connection with foundation design. The corresponding flexural moments at each floor level are important in connection with the calculation of vertical stresses in the columns and walls of the structure. These moments can be computed from modal analyses or from an effective lateral force analysis. Modifications in the base flexural overturning moment can be made by use of the reduction factors given above when the equivalent lateral force method is used. The modal analysis method takes account of these effects directly in the structure, although an additional reduction of 20 percent of the moment may be allowed in the modal analysis for the foundation forces only.

Vertical Component of Ground Motion

In highly seismic zones where the vertical acceleration of the ground may be large compared with the acceleration of gravity, the normal procedure of neglecting vertical accelerations in design may not be appropriate. Generally, the basis for neglecting these vertical accelerations is the fact that building design usually provides for a high factor of safety in the vertical direction. The dynamic vertical response can be estimated by use of a vertical response spectrum to obtain the equivalent vertical forces for which the building must be designed. In making this computation, the response spectrum used for horizontal motion may be used as a basis, but with a scale factor of approximately two-thirds, which is generally the maximum ratio between vertical and horizontal accelerations in most ground movements due to earthquakes.

Elements which are particularly vulnerable to vertical components of ground motion are columns and walls in compression, and especially prestressed beams or other horizontal elements, and cantilevered elements, where the amplification factors for vertical response may be fairly large, or where there is a relatively small factor of safety for reversed or upward accelerations. In these instances, the amplified vertical acceleration as calculated from the response spectrum and from the period or the frequency of the particular element considered should be taken into account in the development of the design forces.

Combined Effects of Horizontal Motions

Since the building responds in both horizontal directions at the same time and the stresses are caused by both motion inputs, as well as by the simultaneous vertical motion, it is necessary to consider the combined effects of the various directions of input earthquake motion. If these motions are computed separately and individually, one may make the combination in one of several ways. For example, let us define the effect at a particular point in a particular element, such as stress, moment, shear, etc., arising from the horizontal earthquake in one direction as F_1 and from the earthquake in the transverse horizontal direction as F_2. Let us also define the same effect arising from the vertical component of ground motion as F_3, in those instances where this cannot be neglected. In general, the combined effects may be computed as the square root of the sums of the squares of the individual effects, where the resultant effect F is given by the relation

$$F = \sqrt{F_1^2 + F_2^2 + F_3^2} \tag{21}$$

When it is considered appropriate to neglect the vertical acceleration effect, F_3 may be taken as zero in Eq. (21).

In some instances it is difficult to use this relationship, especially when the individual effects produce different components of stress, such as a maximum shear from the horizontal motion in the first direction, and a maximum normal stress from the horizontal motion in the second direction. In this case, the quantities to be combined by Eq. (21) are not the same. For these cases, and for all cases as a general approximation, one may use instead the relationship that the effects of the different directions of motion are given approximately by assuming 1.0 times the effect of input motion in one direction, combined with 1/3 of the effects in the other two directions. In order to use this relationship correctly, one must consider the maximum effect as being the one in which the factor of 1.0 is used, or alternatively one should take 1.0 times the inputs for each direction in turn, combined with 1/3 of the inputs for the other two directions. The summation is made for the absolute values of the responses. It can be shown that this relationship is very close to that given by Eq. (21).

Effects of Gravity Loads

The effect of gravity loads, when a structure deflects because of the horizontal motions transmitted to it, is to add a secondary moment owing to the eccentricity in the vertical direction of the gravity loads acting through the lateral deflections corresponding to the horizontal responses. As a first approximation, one may compute the horizontal displacements and the effects of the moments produced by the gravity loads directly. In making this calculation, the total horizontal deflection, including the inelastic portion of it, should be used, rather than the elastic component of the horizontal deflection. Consequently, the computed elastic deflection from the design shears and moments computed by the modal analysis or the equivalent lateral force procedure with a reduced design spectrum, must be increased by the factor μ, the ductility factor, in order to obtain the total deflections and the corresponding total gravity load moments or P-delta effects. A quick estimate of these moments can be made to determine whether the P-delta effect is important.

Unfortunately, when the P-delta effect is important, the above method underestimates the actual displacements because of the additional displacement caused by the additional moments accompanying the increment in deflection from the first step in the calculation described above. In effect, there is a series of corrections to be added, which requires the calculation of the successive increments in deflection caused by the P-delta effect and then the additional moments corresponding thereto in successive stages until convergence is reached.

One can make a very good approximation to this summation by considering the quantity θ as defining the relative increment in moment, stress or deflection, due to the first step in the P-delta calculation, and then computing the final moment by use of the factor $1/(1 - \theta)$ as a multiplier times the effect computed without consideration of the P-delta effect.

EQUIVALENT LATERAL FORCE AND MODAL ANALYSIS METHODS

Limitation of Methods

Following are the most important assumptions common to the equivalent lateral force procedure and the modal analysis procedure:

(1) Forces and deformations can be determined by combining the results of independent analyses of a planar idealization of the building for each horizontal component of ground motion, and including torsional moments determined on an indirect, empirical basis.

(2) Nonlinear structural response can be determined to an acceptable degree of accuracy by linear analysis of the building using the design spectrum for inelastic systems. Both analysis procedures are likely to be inadequate if the dynamic response behavior of the building is quite different from what is implied by these assumptions.

In particular, both methods may be inadequate if the lateral motions in two orthogonal directions and the torsional motions are strongly coupled. Buildings with large eccentricities of the centers of story resistance relative to the centers of floor mass, or buildings with close values of natural frequencies of the lower modes and essentially coincident centers of mass and resistance, exhibit coupled lateral-torsional motions. For such buildings independent analyses for the two lateral directions may not suffice, and at least three degrees of freedom per floor -- two translational motions and one torsional -- should be included in the idealized model. The modal method, with appropriate generalizations of the concepts involved, can be applied to analysis of the model. Because natural modes of vibration will show a combination of translational and torsional motions, in determining the modal maxima it is necessary to account for the fact that a given mode might be excited by both horizontal components of ground motion, and modes that are primarily torsional can be excited by translational components of ground motion. Because natural frequencies of a building with coupled lateral-torsional motions can be rather close to each other, the modal maxima should not be combined in accordance with the root-sum-square formula; instead a more general formula should be employed (See Refs. 11, 12).

The manner of combining the maximum responses due to the two horizontal components of ground motion depends on the correlation between these motions. For earthquakes sufficiently intense to be of practical interest the intensities in all horizontal directions are comparable. It follows that there is little error in assuming that ground motions are uncorrelated in any two orthogonal horizontal directions. With this assumption, the combined response considering both components can be estimated as the square root of the sum of squares of the responses to the two individual components.

The equivalent lateral force procedure and both versions of the modal method -- the simpler version and the general version with the three degrees of freedom per floor mentioned in the foregoing paragraphs -- all are best applicable to analysis of buildings in which ductility demands imposed by earthquakes are expected to be essentially uniformly distributed over the various stories. For such buildings, the maximum ductility allowed for a particular structural system and material may be used in determining the inelastic response spectrum. If ductility demands are expected to be considerably different from one story to the next, a simple approach is to decrease the allowable ductility factors in establishing the inelastic response spectrum resulting in larger design forces. Whereas this simple approach is a step in the right direction, the foregoing analysis procedures may still err systematically on the unsafe side if the ductility demands are concentrated in a few stories of the building. For such buildings, the actual strength properties can be explicitly considered and the distribution of ductility demand determined, perhaps only by a nonlinear response analysis. However, nonlinear numerical analyses may not always be practical, nor lead to reliable results.

Choice Between Methods

Both procedures are based on the same basic assumptions and are applicable to buildings whose dynamic response behavior is in reasonable conformity with the implications of these assumptions. The main difference between the two procedures lies in the magnitude of the base shear and distribution of the lateral forces. Whereas in the modal method the force calculations are based on computed periods and mode shapes of several modes of vibration, in the equivalent lateral force method they are based on an estimate of the fundamental period and simple formulas for distribution of forces which are appropriate for buildings with regular distribution of mass and stiffness over height. In what follows, a criterion to decide whether the equivalent lateral force procedure will be adequate in a particular situation is presented.

It would, in general, be adequate to use the equivalent lateral force procedure for buildings with the following properties: seismic force resisting system has the same configuration in all stories and in all floors; floor masses do not differ by more than, say, 30 percent in adjacent floors; and cross-sectional areas and moments of inertia of structural members do not differ by more than about 30 percent in adjacent stories. For other buildings, the following sequence of steps may be employed to decide whether the modal analysis procedure ought to be used:

1. Compute lateral forces and story shears using the equivalent lateral force procedure.

2. Approximately dimension structural members.

3. Compute lateral displacements of the structure as designed in step 2 due to lateral forces from step 1.

4. Compute new sets of lateral forces and story shears by replacing h_i^k in Eq. (19) with the displacements computed in step 3.

5. If at any story the recomputed story shear (step 4) differs from the corresponding original value (step 1) by more than 30 percent, the structure should be analyzed by the modal analysis procedure. If the difference is less than this value the modal analysis procedure is unnecessary, and the structure should be designed using the story shears obtained in step 4; they represent an improvement over the results of step 1.

This method for determining whether modal analysis should be used is efficient, as well as effective. It requires far less computational effort than the use of the modal analysis procedure, and in general it will detect cases for which results from the equivalent lateral force procedure are in significant error; it will not, however, indicate the need for modal analysis when its application would not significantly improve accuracy.

The seismicity of the area and the potential hazard due to failure of the building should also be considered in deciding whether the equivalent lateral force procedure is adequate. For example, even irregular buildings, that may require modal analysis according to the criterion described, may be analyzed by the equivalent lateral force procedure if they are not located in highly seismic areas and do not house critical facilities necessary for post-disaster recovery or a large number of people.

Soil-Structure Interaction

When a structure is founded within or on a base of soil and/or rock, it interacts with its foundation. The forces transmitted to the structure and the feedback to the foundation are complex in nature, and modify the free-field motions. Methods for dealing with soil-structure interaction have been proposed by a number of writers. These methods involve: (1) procedures similar to those applicable to a rigid block on an elastic half space; (2) finite element or finite difference procedures corresponding to various forcing functions acting on the combined structure-soil complex; (3) substructure modelling techniques which may or may not include use of the direct finite element method. Summaries of some of the factors and uncertainties affecting these calculations are given in Refs. 20 and 21. More advanced techniques are under development, but all methods have yet to be tested and therefore conservative interpretation of the results of analysis is required.

However one makes the calculation, one determines a fundamental frequency and higher frequencies of the soil system which interacts with the structure, and effective damping parameters for the soil system taking into account radiation and material damping. Both of these quantities are necessary in order to obtain rational results. Procedures that emphasize one but not the other cannot give a proper type of interaction.

In general, consideration must be given to the influence of local soil and geologic conditions as affecting the site ground motions, both in terms of intensity and frequency content. Soft soil conditions, for example, may preclude the development of high accelerations or velocities within the foundation materials. Consideration must also be given to the development of unstable conditions such as soil liquefaction, slope instability, or excessive settlements. Further, because of the nature of formation of soil deposits and their lack of uniformity in some situations, in order to carry out meaningful calculations it may be desirable to consider the determination of in-situ properties; in such cases the methods of sampling and testing used to infer these properties need careful consideration. Because of the variations in properties and the difficulty of determining them accurately, some degree of variation in the basic parameters used in the calculations should be taken into account.

Finally, the method of calculation used should avoid as much as possible the introduction of spurious results arising from the calculational technique. For example, it is often necessary to avoid "reflecting" or "hard" boundaries where these do not actually exist.

This entire topic is one that requires the most careful consideration, and additional research and study over the next decade probably will be necessary before definitive recommendations on soil-structure interaction can be developed. In the interim for review and upgrading, it is recommended that great care be taken in assessing the need for such analyses. Careful judgment as to the meaning of the results, in the light of the comments given herein, is required. Reliance on any single method is to be avoided.

The ATC provisions (Ref. 17) take into account soil-structure interaction by an approximate method that permits reduction of base shears from the rigid base values in certain instances.

THE APPLIED TECHNOLOGY COUNCIL PROVISIONS

Historical

Seismic design codes in the United States were initiated in the late 1920's with some relatively simple equivalent static formulas. The development of earthquake code provisions proceeded somewhat intermittently until the Structural Engineers Association of California (SEAOC) in 1959-60 published its Recommended Lateral Force Requirements and Commentary which were applicable to California seismic conditions. The SEAOC provisions recognized that the seismic forces induced in a structure are related to the structure's flexibility or fundamental period. Seismic codes in the United States and in many other countries have since been patterned after the SEAOC provisions. A brief history of codes in the United States is shown in Table 3.

The need for a coordinated effort to review existing requirements and state of knowledge and to develop comprehensive seismic design provisions applicable to all of the country was recognized several years ago. It was realized by design professionals and government representatives that the effort would take many years to complete if performed by volunteer committees as codes have been developed in the past. After numerous detailed discussions ATC started development in November 1974 of comprehensive seismic design provisions for buildings that can be adopted by jurisdictions throughout the United States.

The ATC-3 project (as it is designated) was conducted under a contract with the National Bureau of Standards (NBS) with funding by the National Science Foundation - Research Applied to National Needs program (NSF-RANN) and NBS. The program was a major joint undertaking of the engineering profession and professional community including architects and engineering earthquake scientists, building regulatory officials, code promulgating officials, the research community, and the federal government. The project is part of the Cooperative Federal Program in Building Practices for Disaster Mitigation initiated in the spring of 1972 under the leadership of NBS.

The 85 participants involved in the development of the provisions were organized into five working groups composed of 14 task committees, two advisory groups, one independent review group, and a project executive panel.

Two working drafts were submitted to outside review. The February 1976 draft and the January 1977 draft were reviewed by several hundred reviewers. The reviewers represented practice, industry, professional organizations, government agencies, code promulgating groups, and universities. The numerous comments received for each draft were reviewed by the various task committees, and changes and clarifications were made as deemed appropriate in the final report.

It is intended that the provisions can be used by model code groups and government agencies to develop seismic design regulations for buildings. However, the participants have recommended that the provisions be tested before they are placed in the code promulgating process. A program is underway to test the provisions by having practicing professionals make comparative designs of various types of buildings in different areas of the United States.

Basic Concepts of ATC Code

The primary basis for development of the seismic design provisions for buildings was to protect life safety, and to ensure continued functioning of essential facilities needed during and after a catastrophe. It was realized that zero risk is not realisitically possible or feasible; expenditures to obtain absolute safety (if it were possible) may not be desirable, as the resources to construct buildings are limited and society must decide how it will allocate the available resources among the various ways in which it desires to protect life safety.

In the development of the U. S. seismic codes currently in use, it was recognized that the specified design forces are considerably smaller than those that might be encountered in moderate or major earthquakes. Primary consideration is given in the design for seismic resistance to the main structural framing system(s). Consideration is given either explicitly or implicitly to the effects (both good and bad) of interior partitions, exterior covering, different types of materials, and damping.

However this approach has caused misunderstanding that can result in building designs with serious deficiencies such as inadequate connections or tying together of building components and inadequate provision(s) for occurrence of building deformation in excess of those calculated for the design forces.

The ATC-3 provisions were intended to be logically based with explicit consideration given to factors that are generally implicit in present code design provisions. A number of new concepts which are significant departures from existing seismic building codes are included, such as:

1. More realistic seismic ground motion intensities.

2. Consideration of the effects of distant earthquakes on long period buildings.

3. Response modification coefficients (reduction factors) which are based on consideration of the inherent capacity for energy absorption, damping associated with inelastic response, and observed past performance of various types of framing systems.

4. Complexity of analysis and design dependent on importance or use factor, assigned building seismic performance category, and seismic motion intensity.

5. Simplified seismic response coefficient formulas related to fundamental period of building but with certain restrictions.

6. Detailed requirements for architectural, electrical, and mechanical systems, and components.

7. Material design stresses approaching yield.

8. Guidelines for assessment and systematic abatement of seismic hazards in buildings.

9. Guidelines for assessment of earthquake damage, and strengthening or repair of damaged buildings and potential seismic hazards in existing buildings.

It should be noted that the provisions are intended to apply only to buildings and do not contain design requirements for special structures such as bridges, transmission towers, offshore structures, piers and wharves, industrial towers and equipment, and nuclear reactors.

Seismic Ground Motion Intensities

Before developing the seismic risk maps and corresponding ground motion intensities, several factors were considered. As noted previously, current U. S. earthquake design codes specify seismic forces that are smaller than those anticipated. It was therefore decided that (1) a realistic appraisal should be made of the expected ground motion intensities and (2) the probability of the design ground shaking being exceeded should be approximately the same in all parts of the country. This implies that the design ground shaking is not necessarily the most intense motion that might conceivably occur at a location. It is possible that the design earthquake ground shaking might be exceeded, although the probability of this happening is quite small. The zoning maps currently in use in the United States have been based upon estimates of the maximum ground shaking during recorded history without consideration of the frequency of occurrence of earthquakes.

It was also agreed that the relationships between design lateral forces and the period of a structure should consider the distance from anticipated earthquake sources. It has been observed that the higher frequencies in ground motion attenuate more rapidly with distance than the lower frequencies. Thus flexible structures at a distance from an earthquake may be significantly affected whereas stiff structures may not. This is the basic reason for preparing two separate ground motion parameter maps.

A third decision was that areas would not be microzoned or actual faults located on the maps, and variations of ground shaking over short distances of ten miles or less would not be considered. Microzoning should be done by experts who are familiar with local conditions.

The recommended seismic design regionalization maps used in the Code are based on an evaluation of historic seismicity, frequency of occurrence of the earthquake motions and underlying geology where possible. They reflect the collective judgment of several committees based upon the best data available in 1976, adjusted and tempered by experience and judgment. It is expected that the maps and coefficients will change with time as more knowledge is gained about earthquakes and their ground motions and as society better understands the establishment of an acceptable risk.

Seismic Design Coefficients

The effective peak acceleration A_a and effective peak velocity-related acceleration A_v of the Code should be considered as normalizing factors for construction of smoothed elastic response spectra for ground motions of normal duration. A_a is proportional to spectral ordinates for periods in the range of 0.1 to 0.5 sec, while A_v is proportional to spectral ordinates of a period of about 1.0 sec. It should be emphasized that A_a and A_v values are considered to be appropriate for normal building design. Motions in high seismic areas near active faults could exceed these values, especially in locations inside of the 0.4 g contour.

The effects of distant earthquakes on flexible structures have been considered for only a few structures. It is generally recognized that the relationships between design lateral forces and period of a structure should take into account distance from probable earthquake sources. Higher frequencies attenuate more rapidly with distance than the lower frequencies. At distances greater than 100 km from the source, flexible structures may be significantly affected, while stiff structures are not. The A_v map was therefore constructed to consider the effects of distant earthquakes.

Design Ground Shaking

The best presently available tool for describing the design ground shaking is a smoothed elastic response spectrum for single-degree-of-freedom systems. Such a spectrum provides a quantitative description of both the intensity and frequency content of a ground motion. Smoothed elastic response spectra for 5 percent damping were used for development of the maps and for the inclusion of the effects of local ground conditions. A smoothed elastic response spectrum is not necessarily the best means for describing the design ground shaking. A set of acceleration time histories, say four or more, whose average elastic response spectrum is similar to the design spectrum would be better for buildings and structures of special importance. Such an approach is not feasible for the vast majority of buildings.

The response spectrum has one major deficiency in that it does not reflect the duration of the shaking. The major effect of duration is the possible loss of strength once a structure yields. Duration effects were not considered explicitly in drawing up the recommended provisions; however, the possibility that the design motion might be longer in highly seismic areas and shorter in less seismic areas influenced the assignment of the seismicity index which is used in the provisions.

General Comments

An abridged summary of the more important aspects of the ATC Tentative Provisions is contained in Tables 4 through 10. These tables are self explanatory in general. Table 8 lists the spectral reduction factors R which depend in part on judgment, and the deflection coefficient C_d which is essentially the same as the ductility factor.

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TABLE 1. SPECTRUM AMPLIFICATION FACTORS FOR HORIZONTAL ELASTIC RESPONSE

Damping, % Critical	One Sigma (84.1%)			Median (50%)		
	A	V	D	A	V	D
0.5	5.10	3.84	3.04	3.68	2.59	2.01
1	4.38	3.38	2.73	3.21	2.31	1.82
2	3.66	2.92	2.42	2.74	2.03	1.63
3	3.24	2.64	2.24	2.46	1.86	1.52
5	2.71	2.30	2.01	2.12	1.65	1.39
7	2.36	2.08	1.85	1.89	1.51	1.29
10	1.99	1.84	1.69	1.64	1.37	1.20
20	1.26	1.37	1.38	1.17	1.08	1.01

TABLE 2. RECOMMENDED DAMPING VALUES

Stress Level	Type and Condition of Structure	Percentage of Critical Damping
Working stress, no more than about 1/2 yield point	a. Vital piping or equipment	1 to 2
	b. Welded steel, prestressed concrete, well reinforced concrete (only slight cracking)	2 to 3
	c. Reinforced concrete with considerable cracking	3 to 5
	d. Bolted and/or riveted steel, wood structures with nailed or bolted joints	5 to 7
At or just below yield point	a. Vital piping or equipment	2 to 3
	b. Welded steel, prestressed concrete (without complete loss in prestress)	5 to 7
	c. Prestressed concrete with no prestress left	7 to 10
	d. Reinforced concrete	7 to 10
	e. Bolted and/or riveted steel, wood structures, with bolted joints	10 to 15
	f. Wood structures with nailed joints	15 to 20

TABLE 3

SEISMIC DESIGN CODES IN THE UNITED STATES

POST 1906	SAN FRANCISCO REBUILT TO 30 PSF WIND	
1927	UNIFORM BUILDING CODE ($C = 0.075$ to 0.10)	
1933	LOS ANGELES CITY CODE ($C = 0.08$)	
1943	LOS ANGELES CITY CODE ($C = \frac{60}{N + 4.5}$, $N \leq 13$ STORIES)	
1952	ASCE-SEAONC ($C = \frac{K_1}{T}$, $K_1 = 0.015 - 0.025$)	
1959	SEAOC $V = KCW$, $C = \frac{0.05}{\sqrt[3]{T}}$	
1974	SEAOC	$V = ZIKCSW$
1976	UBC	$V = ZIKCSW$
1977	ATC-3 TENTATIVE RECOMMENDATIONS	

TABLE 4

ATC-3 PROVISIONS

- APPLICABLE TO THE ENTIRE UNITED STATES
- PROTECT LIFE SAFETY
- ENSURE FUNCTIONING OF ESSENTIAL FACILITIES
- ALLOW FOR INGENUITY OF THE DESIGNER
- APPLICABLE TO NEW AND EXISTING BUILDINGS
- INCLUDE STRUCTURAL AND NONSTRUCTURAL ELEMENTS
- APPLICABLE ONLY TO BUILDINGS

TABLE 5
ATC-3 CONCEPTS

- REALISTIC GROUND MOTION INTENSITIES
- DISTANT EARTHQUAKE EFFECTS
- RESPONSE MODIFICATION COEFFICIENTS
- ANALYSIS AND DESIGN DEPENDENT ON
  - SEISMICITY INDEX
  - IMPORTANCE OR USE
  - BUILDING SEISMIC PERFORMANCE
- SIMPLIFIED BUILDING PERIOD CALCULATION
- NONSTRUCTURAL SYSTEMS AND COMPONENTS
- STRENGTH DESIGN RATHER THAN WORKING STRESS
- SEISMIC HAZARDS IN EXISTING BUILDINGS
- ASSESSMENT OF EARTHQUAKE DAMAGE

TABLE 6
ATC-3 DESIGN STEPS-(I)

- LOCATE SITE
- MAP AREA NUMBER, A_a AND A_v
- TABLE I — SEISMICITY INDEX
- TABLE 2 — SEISMIC HAZARD EXPOSURE GROUP AND CATEGORY
- REVIEW DESIGN REQUIREMENTS
  - SELECT SOIL PROFILE
  - FRAMING SYSTEM
  - PERFORMANCE CATEGORY (SEE TABLE II)
  - SELECT ANALYSIS PROCEDURE
- ANALYSIS
  - NO DESIGN (WIND GOVERNS)
  - EQUIVALENT LATERAL FORCE METHOD (ELF)
  - MODAL ANALYSIS

TABLE 7
ATC-3 DESIGN STEPS, ELF ANALYSIS

$$V = C_s W$$

$$C_s = \frac{1.2\, A_v S}{R T^{2/3}}$$

$$C_s = \frac{2.5\, A_a}{R}$$

$$C_s = \frac{2.0\, A_a}{R}, \quad (S_3 \text{ SOIL}, \; A_a \geq 0.3)$$

$$T \leq 1.2\, T_a$$

$$T_a = C_R h_n^{3/4}, \quad C_R = 0.035 \text{ STEEL FRAME}$$
$$= 0.025 \text{ RC FRAME}$$

$$T_a = \frac{0.05\, h_n}{\sqrt{L}}, \quad \text{ALL OTHERS}$$

S = 1.0 FOR ROCK OR STIFF SOIL
1.2 FOR DEEP SOIL SITES
1.5 FOR SOFT SOILS

TABLE 8

RESPONSE MODIFICATION COEFFICIENT R AND DEFLECTION C_d

TYPE OF STRUCTURAL SYSTEM	VERTICAL SEISMIC RESISTING SYSTEM	COEFFICIENT R	C_d
BEARING WALL SYSTEM	LIGHT WALLS, SHEAR PANELS	$6\frac{1}{2}$	4
SEISMIC RESISTANCE			
SHEAR WALLS OR BRACED FRAMES	SHEAR WALLS		
	RC	$4\frac{1}{2}$	4
	RM	$3\frac{1}{2}$	3
	BRACED FRAMES	4	$3\frac{1}{2}$
	SHEAR WALLS -- URM	$1\frac{1}{4}$	$1\frac{1}{4}$
BUILDING FRAME SYSTEM	LIGHT WALLS, SHEAR PANELS	7	$4\frac{1}{2}$
SEISMIC RESISTANCE	SHEAR WALLS		
	RC	$5\frac{1}{2}$	5
SHEAR WALLS OR BRACED FRAMES	RM	$4\frac{1}{2}$	4
	BRACED FRAMES	5	$4\frac{1}{2}$
	SHEAR WALLS -- URM	$1\frac{1}{2}$	$1\frac{1}{2}$
MOMENT RESISTING FRAME SYSTEM	SMF		
	STEEL	8	$5\frac{1}{2}$
	RC	7	6
SEISMIC RESISTANCE -- ORDINARY OR SPECIAL MOMENT FRAMES			
	OMF		
	STEEL	$4\frac{1}{2}$	4
	RC	2	2
DUAL SYSTEM	SHEAR WALLS		
	RC	8	$6\frac{1}{2}$
	RM	$6\frac{1}{2}$	$5\frac{1}{2}$
	WOOD SHEATHED SHEAR PANELS	8	5
	BRACED FRAMES	6	5
INVERTED PENDULUM STRUCTURES	SMF		
	STRUCT. STL.	$2\frac{1}{2}$	$2\frac{1}{2}$
	RC	$2\frac{1}{2}$	$2\frac{1}{2}$
	OMF		
	STRUCT. STL.	$1\frac{1}{4}$	$1\frac{1}{4}$

TABLE 9
ATC-3 DESIGN STEPS (2)

- VERTICAL DISTRIBUTION OF SHEAR

$$F_x = V \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

- HORIZONTAL DISTRIBUTION
  - RELATIVE RIGIDITY OF RESISTING ELEMENTS
  - TORSIONAL MOMENTS
  - ACCIDENTAL TORSION
- OVERTURNING

TABLE 10
ATC-3 DESIGN STEPS (3)

- REDUNDANCY
- DISCONTINUITIES IN STRENGTH
- DISCONTINUITIES IN STIFFNESS
- STORY DRIFT
- P-DELTA EFFECTS
- REVIEW DESIGN (AND REVISE)
- MAKE DYNAMIC ANALYSIS
- FINAL DESIGN AND DETAILS
- QUALITY ASSURANCE PLAN

TABLE 11
SEISMIC PERFORMANCE CATEGORY REQUIREMENTS

A. BUILDING PARTS MUST BE INTERCONNECTED

B. MINIMUM SEISMIC DESIGN REQUIRED

C. FRAMING LIMITATIONS FOR BUILDINGS OVER 160 FT
   - DUCTILE FRAME
   - DUAL SYSTEM
   - REDUNDANCY FOR BRACED FRAMES OR SHEAR WALLS FOR BUILDINGS OVER 240 FT

D. HEIGHT LIMITATION 100 FT AND 160 FT

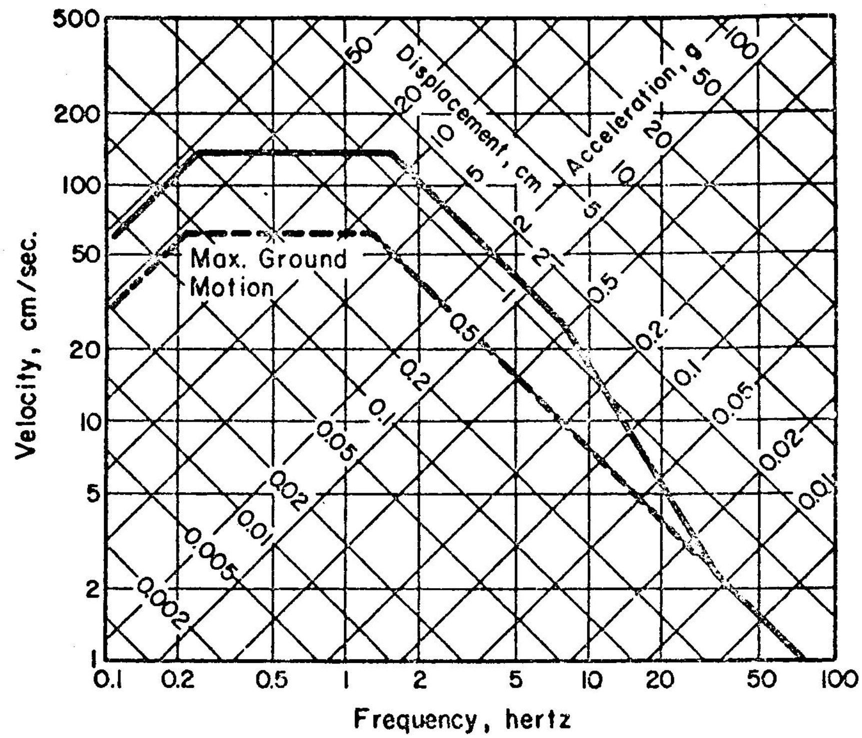

FIG. I ELASTIC DESIGN SPECTRUM, HORIZ. MOTION, (0.5g MAX. ACCEL., 5 % DAMPING, ONE SIGMA CUMULATIVE PROBABILITY)

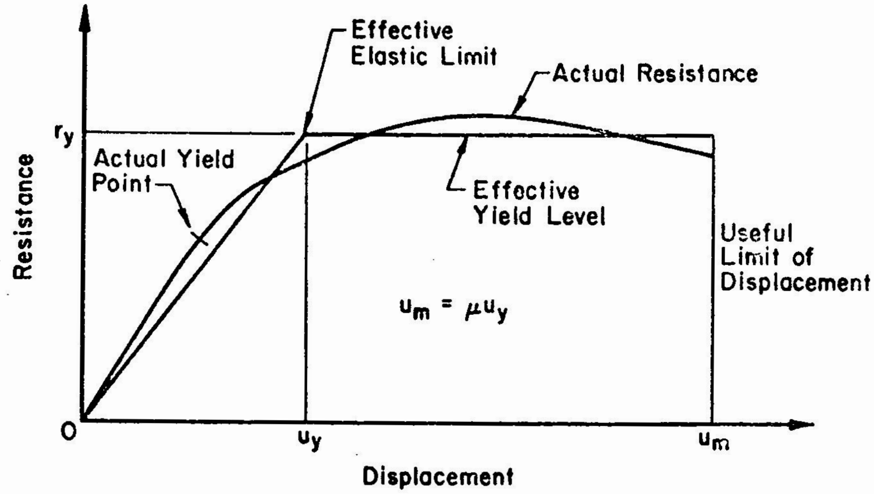

FIG. 2 RESISTANCE – DISPLACEMENT RELATIONSHIP

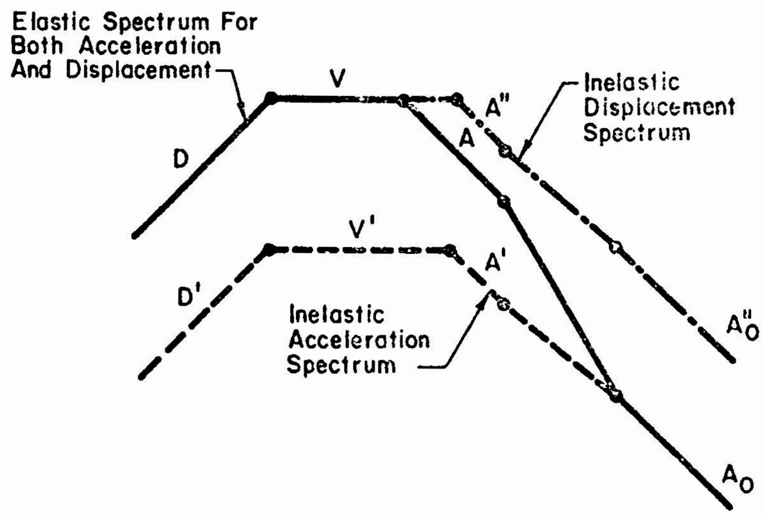

FIG. 3 DESIGN SPECTRA FOR EARTHQUAKES

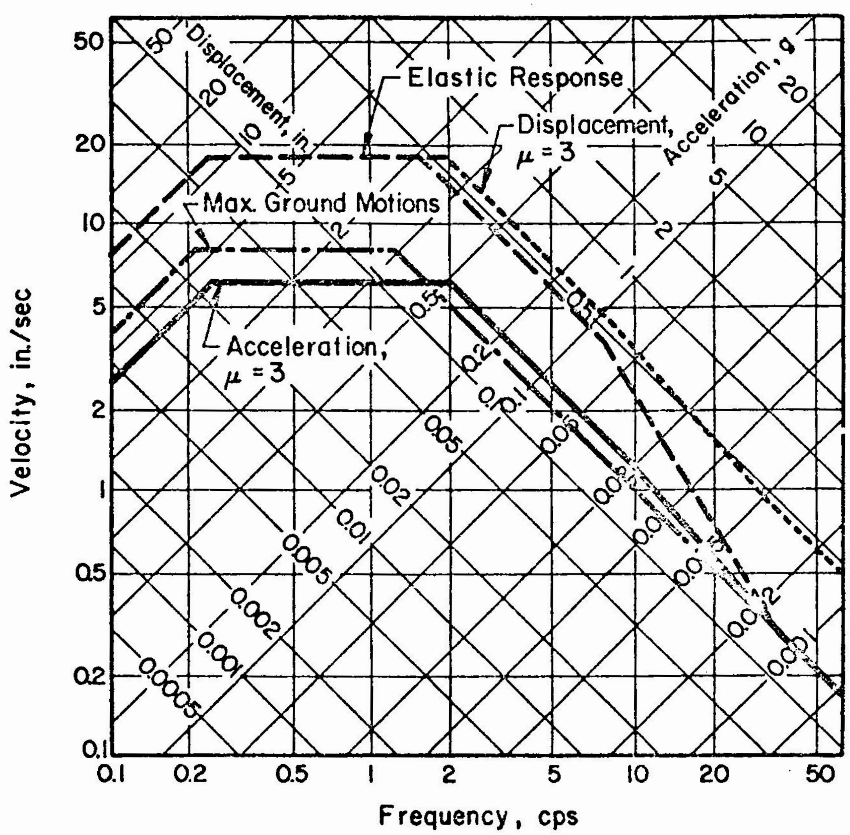

FIG. 4 METHOD OF DRAWING ELASTIC AND INELASTIC DESIGN SPECTRA