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NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings

Part 2
Commentary

EARTHQUAKE HAZARDS REDUCTION SERIES 18

BUILDING SEISMIC SAFETY COUNCIL

The Building Seismic Safety Council (BSSC) is an independent, voluntary body that was established under the auspices of the National Institute of Building Sciences (NIBS) in 1979 as a direct result of nationwide interest in the seismic safety of buildings. Its membership (see inside back cover) represents a wide variety of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings. To fulfill its purpose, the BSSC:

- Promotes the development of seismic safety provisions suitable for use throughout the United States;
- Recommends, encourages, and promotes the adoption of appropriate seismic safety provisions in voluntary standards and model codes;
- Assesses progress in the implementation of such provisions by federal, state, and local regulatory and construction agencies;
- Identifies opportunities for improving seismic safety regulations and practices and encourages public and private organizations to effect such improvements;
- Promotes the development of training and educational courses and materials for use by design professionals, builders, building regulatory officials, industry representatives, other members of the building community, and the public;
- Advises government bodies on their programs of research, development, and implementation; and
- Periodically reviews and evaluates research findings, practices, and experience and makes recommendations for incorporation into seismic design practices.

The BSSC's area of interest encompasses all building-type structures and includes explicit consideration and assessment of the social, technical, administrative, political, legal, and economic implications of its deliberations and recommendations. It believes that the achievement of its purpose is a concern shared by all in the public and private sectors; therefore, its activities are structured to provide all interested entities (for example, government bodies at all levels, voluntary organizations, business, industry, the design profession, the construction industry, the research community, and the general public) with the opportunity to participate. The BSSC also believes that the regional and local differences in the nature and magnitude of potentially hazardous earthquake events require a flexible approach to seismic safety that allows for consideration of the relative risk, resources, and capabilities of each community.

The BSSC is committed to continued technical improvement of seismic design provisions, assessment of advances in engineering knowledge and design experience, and evaluation of earthquake impacts. It recognizes that appropriate earthquake hazard reduction measures and initiatives should be adopted by existing organizations and institutions and incorporated, whenever possible, into their legislation, regulations, practices, rules, codes, relief procedures, and loan requirements so that these measures and initiatives become an integral part of established activities, not additional burdens. The BSSC itself assumes no standards-making and/or -promulgating role; rather, it advocates that standards-formulation organizations consider BSSC recommendations for inclusion into their documents and standards.

BSSC Program on Improved Seismic Safety Provisions

NEHRP
(National Earthquake Hazards Reduction Program)
RECOMMENDED PROVISIONS
FOR THE
DEVELOPMENT OF
SEISMIC REGULATIONS FOR NEW BUILDINGS

1988 Edition

PART 2

COMMENTARY

Prepared by the
Building Seismic Safety Council
for the
Federal Emergency Management Agency

Based on the Applied Technology Council's
1978 *Tentative Provisions for the Development of Seismic Regulations for Buildings*

BUILDING SEISMIC SAFETY COUNCIL
Washington, D.C.
1988

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Building Seismic Safety Council reports include the following:

Abatement of Seismic Hazards to Lifelines: Proceedings of the Building Seismic Safety Council Workshop on Development of an Action Plan, 6 volumes, 1987

Action Plan for the Abatement of Seismic Hazards to New and Existing Lifelines, 1987

Guide to Use of the NEHRP Recommended Provisions in Earthquake-Resistant Design of Buildings, 1985 Edition, 1987

Improving the Seismic Safety of New Buildings: A Community Handbook of Societal Implications (Revised Edition) and *Societal Implications: Selected Readings*, 1986

Improving the Seismic Safety of New Buildings: A Non-Technical Explanation of the NEHRP Recommended Provisions, 1986

NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings, 1988 Edition, 1988

Seismic Considerations: Elementary and Secondary Schools, 1987

Seismic Considerations: Health Care Facilities, 1987

Seismic Considerations: Hotels and Motels, 1987

Seismic Considerations: Apartment Buildings, 1988

Seismic Considerations: Office Buildings, 1988

For further information concerning any of these documents, contact the Executive Director, Building Seismic Safety Council, 1015 15th St., N.W., Suite 700, Washington, D.C. 20005.

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NOTE

Those portions of the 1988 Edition of the *Provisions* that are substantively different from the 1985 Edition are identified in the margins as follows:

Additions or Revisions

Deletions

Not highlighted are editorial changes and the terminology changes required because of the change in Seismic Performance Category designations (from A, B-1, B-2, C, and D to A, B, C, D, and E) and the replacement of the "Seismicity Index" with explicit citations of velocity-related acceleration (A_v).

A summary of the differences between the 1985 and 1988 Edition of the *Provisions* is presented as Appendix B of this *Commentary* volume.

Chapter 1 Commentary

GENERAL PROVISIONS

Chapter 1 provides general requirements for applying the analysis and design provisions contained in Chapters 3 through 12 of the *NEHRP Recommended Provisions*. It is similar to what might be incorporated in a code as administrative regulations.

Although Chapter 1 is designed to be as compatible as possible with normal code administrative provisions (especially as exemplified by the three national model codes), it is written as the guide to use of the rest of the document, not as a regulatory mechanism. The word "shall" is used in the *Provisions*, not as a legal imperative, but simply as the language necessary to ensure fulfillment of all the steps necessary to technically meet a minimum standard of performance.

It is important to note that the *NEHRP Recommended Provisions* is intended to serve as a **source** document for use by any interested member of the building community. Thus, some users may alter certain information within the *Provisions* (e.g., the determination of which use groups are included within the higher Seismic Hazard Exposure Groups might depend on whether the user concluded that the generally more-demanding design requirements were necessary). It is strongly emphasized, however, that such "tailoring" should be carefully considered by highly qualified individuals who are fully aware of all the implications of any changes on all affected procedures in the analysis and design sequences of the document.

Further, it should be remembered that the *NEHRP Recommended Provisions* is national in scope and presents minimum criteria. It is not intended to, nor does it justify, any reduction in higher standards that have been locally established, particularly in areas of highest seismicity.

Reference is made throughout the document to decisions and actions that are delegated to unspecified authorities referred to as the "Regulatory Agency." The document is intended to be applicable to many different types of jurisdictions and chains of authority, and an attempt has been made to recognize situations where more than technical decision-making can be presumed. In fact, the document anticipates the need to establish standards and approval systems to accommodate the use of the document for development of a regulatory system. A good example of this is in Sec. 1.5, "Alternate Materials and Methods of Construction," where the need for well-established criteria and systems of testing and approval are recognized even though few such systems are in place. In some instances, the decision-making mechanism referred to is clearly most logically the province of a building official or department; in others, it appears that the authority may be a law-making body such as a legislature or city council; in still others, the decisions may be the province of a state or local policy-making body. The term "Regulatory Agency" has been used to apply to all of these entities.

A good example of the need of keeping such generality in mind is provided by the California law concerning the design and construction of schools. That law establishes requirements for independent special inspection approved and supervised by the Office of the State Architect, a state-level office that does not exist in many other states.

1.1 PURPOSE

The goal of the *NEHRP Recommended Provisions* is to present criteria for the design and construction of buildings subject to earthquake ground motions in order to minimize the hazard to life for all buildings, to increase the expected performance of higher occupancy structures as compared to ordinary structures, and to improve the capability of essential facilities to function during and after an earthquake. The primary function of the *Provisions* is to provide the minimum criteria considered prudent and economically justified for the protection of life safety in buildings subject to earthquakes at any location in the United States. The *Provisions* have been extensively reviewed and balloted by the building community and, therefore, are a proper source for the development of building codes in areas of seismic exposure.

Some design standards go farther than these provisions and attempt to minimize damage as well as protect building occupants. For example, Title 17 and Title 21 of California's *Administrative Code* have added property protection in relation to the design and construction of hospitals and public schools. The *NEHRP Recommended Provisions* document generally considers property damage as it relates to occupant safety for ordinary buildings. For high occupancy and essential facilities, the damage limitation criteria are more strict in order to better provide for the safety of occupants and the continued functioning of the facility. Some structural and nonstructural damage can be expected as a result of the "design ground motions" because these provisions allow inelastic energy dissipation by utilizing the deformability of the structural system. For ground motions in excess of the design levels, the intent is that there be a low likelihood of collapse.

It must be emphasized that absolute safety and no damage even in an earthquake event with a reasonable probability of occurrence cannot be achieved economically. The objective of these provisions is therefore to present the minimum requirements to provide reasonable and prudent life safety for building occupants. For most structures designed and constructed according to the *Provisions*, it is expected that structural damage from even a major earthquake would likely be repairable; however, this would depend upon a number of factors including the type, materials, and details of construction actually used.

Because of the complexity of and the great number of variables involved in seismic design (e.g., the dynamic characteristics of the structure and the variability in ground motion, intensity of the earthquake, distance to the epicenter of the seismic disturbance, and soil type), these provisions detail only the minimum criteria in general terms. Thus, the experienced structural engineer is relied upon to exercise judgment in interpreting and adapting the basic principles to a specific project.

The *Provisions* are applicable in all sections of the United States exposed to earthquake ground motions because the "design earthquake" ground motions are based on an estimated 90 percent probability of not being exceeded in a 50-year period. This is in keeping with that provided for other natural hazards such as wind, snow, and floods. However, it must be emphasized that larger earthquakes are possible and may occur during the life of a structure.

In some areas, the "probable" and the "maximum intensity" earthquake are approximately the same, but this is not true in many other earthquake-prone parts of the country. In the central and eastern United States, the "maximum intensity" earthquake often may be two or more times larger than the "probable" earthquake. Although the probability of the "maximum" event's occurring during a structure's life is very small, it can nevertheless occur at any time and most certainly will occur sometime in the future. In order to quantify this possibility, two sets of maps are presented, one set giving accelerations and velocities with 90 percent probabilities of not being exceeded in 50 years and another set giving accelerations and velocities with 90 percent probabilities of not being exceeded in 250 years. Use of these maps will help regulatory agencies to rationally appraise the possibility that larger earthquakes will occur and to modify the *Provisions* accordingly. Alternative actions could include obtaining a specific site evaluation, ignoring the recommended "cap" level, or using the longer structural life risk level map as the case may be.

Where damage control is desired, the design must provide not only sufficient strength to resist the specified seismic loads but also must provide the proper stiffness to limit the lateral deflection. Damage to nonstructural elements may be minimized by proper limitation of deformations; by careful attention to detail; and by providing proper clearances for exterior cladding, glazing, partitions, and wall panels. The nonstructural elements can be separated or floated free and allowed to move independently of the structure. If they are tied rigidly to the structure, these elements should be protected from deformations that can cause cracking; otherwise, one must expect such damage. It should be recognized, however, that major earthquake ground motions can cause deformations much larger than the specified drift limits in these provisions.

Where prescribed wind loading governs the stress or drift design, the resisting system must still conform to the special requirements for seismic systems. This is required in order to resist, in a ductile manner, potential seismic loadings in excess of the prescribed loads.

The proper continuous load path is an obvious design requirement for equilibrium, but experience has shown that it often is overlooked and that significant damage and collapse can result. The basis for this design requirement is twofold:

- To ensure that the design has fully identified the particular lateral force resisting system and its appropriate design level and

- To allow the design basis to be fully identified for the purpose of future modifications or changes in the structure.

Detailed requirements for selecting or identifying and designing this load path are given in the appropriate design and materials chapters.

1.2 SCOPE

The scope statement establishes in general terms the applicability of the *Provisions* as a base of reference. Certain buildings are exempted and need not comply:

- Buildings for agricultural use are generally excepted by most regulations from code requirements because of the exceptionally low risk to life involved.

- Normal one- and two-family dwellings in areas with the coefficient A_v less than 0.15 (v less than 12 for the "Appendix to Chapter 1") are excepted because they represent exceptionally low risks (see Sec. 1.4.1).

Existing buildings, except additions thereto or changes of occupancy therein, are not within the scope of the *Provisions*. FEMA is currently (1988) sponsoring work on the mitigation of the seismic hazard to existing buildings; for information, write FEMA, Earthquake Programs, Washington, D.C. 20472.

Many other types of structures require seismic design procedures that are beyond the scope of these provisions. Structures such as power plants, bridges, dams, retaining walls, docks, and off-shore platforms require special design criteria. When a particular structure is not within a group treated by these provisions, the structural engineer must establish criteria to suit the special requirements for performance and reliability.

These provisions are not written to prevent damage due to earth slides (such as those that occurred in Anchorage, Alaska) or to liquefaction (such as occurred in Niigata, Japan). They provide for only minimum required resistance to earthquake ground-shaking, without settlement, slides, subsidence, or faulting in the immediate vicinity of the structure.

1.3 APPLICATION OF PROVISIONS

The requirements for application of the provisions in Chapters 2 through 12 to new buildings, additions to existing buildings, and change of use are established in this section.

1.3.1 New Buildings

A simple procedure is established for one- and two-story wood frame dwellings in regions of higher seismicity. Although some control is necessary to ensure the integrity of such structures, it is felt that the requirements of Sec. 9.3 and 9.7 are adequate to provide the safety required based on the history of such frame construction--especially low structures--in earthquakes.

1.3.2 Additions to Existing Buildings

Requirements for additions--both horizontal and vertical--are written on the basis that the *Provisions* do not include criteria for alterations and repairs to existing buildings.[1]

This section has been included to cover specifically the cases where additions are made to existing buildings. The intent is that the addi-

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[1]The 1985 Edition of these *NEHRP Recommended Provisions* included a third part that presented provisions covering existing buildings. These provisions were developed by the Applied Technology Council (ATC) and published in ATC Report 3-06, *Tentative Provisions for the Development of Seismic Regulations for New Buildings* (1978). They were included with the 1985 Edition only as guidance for those interested in existing buildings.

tion as well as the existing building be made to comply with the *Provisions* unless the addition is structurally independent of the existing building. Where the addition is not independent of the existing building, this section permits an increase of up to 5 percent of the mass contributing to seismic forces in any elements of the existing building without bringing the entire building into conformance with this document.

1.3.3 Alterations, Repairs, and Change of Use

Although the *Provisions* do not apply to the alteration or repair of existing buildings, it is strongly recommended that changes to an existing building:

1. Should not reduce the lateral force resistance of the building,
2. Should provide for the seismic forces required by the *Provisions*, or
3. Should comply with legally adopted provisions regulating the repair and rehabilitation of existing buildings as related to earthquake resistance.

When a change in use results in a change to a higher Seismic Hazard Exposure Group, the building must be made to conform to the *Provisions* for the new Seismic Hazard Exposure Group.

1.4 SEISMIC PERFORMANCE

The requirements for analysis and design of buildings presented in the *Provisions* are based on a seismic hazard criterion that reflects the relationship between the use of the building and the level of shaking to which it may be exposed. This relationship primarily reflects concern for life safety and, therefore, the degree of exposure of the public to hazard based on the measure of risk.

The purpose of Sec. 1.4.1 and 1.4.2 is to provide the means for establishing a measure of seismic risk for a building of any use group and in any area of the United States. Based on this measure, the key to the application of the *Provisions*, including when quality assurance procedures are required (Sec. 1.6), is identified. This key is the Seismic Performance Category of Table 1-2 (Table 1A-2 for the "Appendix to Chapter 1").

1.4.1 Design Ground Motions

This portion of the *Commentary* provides the background for Sec. 1.4.1 as well as for the seismic design coefficient, C_s, in Sec. 4.2.

There are several reasons why the earthquake ground-shaking for design cannot be achieved solely by following an agreed-upon set of scientific principles:

1. The causes of earthquakes are not yet fully understood, and experts do not agree on how the knowledge that is available should be interpreted to specify ground motions for use in design.

2. To achieve workable building code provisions, it is necessary to simplify greatly the enormously complex matter of earthquake occurrence and ground motions.

3. Any specification of design ground-shaking implies a balancing of the risk of that motion's occurring against the cost to society of requiring that structures be designed to withstand that motion.

Hence, judgment, engineering experience, and political wisdom are as necessary as science. In addition, the design ground-shaking does not by itself determine how a structure will perform during a future earthquake; there must be a balance between the specified shaking and the rules used to translate that shaking into a design.

The recommended regionalization maps and seismic design coefficients are the result of the collective judgment of several committees that prepared the original 1978 ATC report, based upon the best scientific knowledge available in 1976, adjusted and tempered by experience and judgment. It was expected, however, that the maps and coefficients would change with time as the profession gained more knowledge about earthquakes and their resulting ground motions and as society gained greater insight into the process of establishing acceptable risk. The first significant such changes are included in the "Appendix to Chapter 1" in the 1988 Edition of the *Provisions*. This appendix, which includes new maps and necessary adjustments in coefficients for the use of those maps, is discussed in the final section of this "Chapter 1 Commentary." The remainder of this section strives to explain the bases for the various original recommendations as a guide both to the user of the *Provisions* and to those who will continue to improve the *Provisions* in the future; it does not address the "Appendix to Chapter 1."

Policy Decisions

The recommended ground-shaking regionalization maps are based on several policy decisions, the first two of which are departures from past practice in the United States.

The first decision was that the distance from anticipated earthquake sources should be taken into account. This decision reflects the observation that the higher frequencies in ground motion attenuate more rapidly with distance than the lower frequencies. Thus, at distances of 100 km or more from a major earthquake, flexible buildings may be more seriously affected than stiff buildings. To accomplish the objective of this policy decision, it proved necessary to use two separate ground motion parameters and, therefore, to prepare a separate map for each.

The second policy decision affecting the maps was that the probability of exceeding the design ground-shaking should be roughly the same in all parts of the country. Thus, the *NEHRP Recommended Provisions* maps are different from other zoning maps used in the United States that have been based on estimates of the maximum ground-shaking experienced during the recorded historical period without consideration of how frequently such motions might occur. There is not unanimous agreement in the profession with this policy decision. In part, this lack of agreement reflects doubt as to how well the probability of ground motion occurrence can be estimated with today's knowledge and disagreement with the specific procedures used to make the estimates rather than any true disagreement with the goal. Further, it really is the probability of structural failures with resultant casualties that is of concern, and the geographical distribution of that probability is not necessarily the same as the distribution of the probability of exceeding some ground motion. (This point is discussed further below under "Implied Risk.") Thus, the goal as stated is not necessarily the ideal goal but is judged to be the most workable goal for the present time.

The second policy decision implies that the design ground-shaking is not necessarily the most intense motion that might conceivably occur at a location. This is not a new idea for past codes implied the same thing; however, it does seem wise to state the matter very clearly: It is possible that the design earthquake ground-shaking might be exceeded during the life of the structure--although the probability of this happening is quite small. In this respect, several points must be emphasized:

- Considering the significant cost of designing a structure for extreme ground motions, it is undesirable to require such a design unless there is a significant probability that the extreme motion will occur or unless there is a particularly severe penalty associated with the failure or nonfunctioning of the structure.

- A building properly designed for a particular ground motion will provide considerable protection to the lives of occupants during a more severe ground motion.

- Even if it were desirable to design for the "extreme" or "maximum credible" ground motion, it is not yet possible to get agreement on how intense this motion might be. This is especially true for the less seismic portions of the country.

The third important policy decision, which also is not new, was that the regionalization maps should not attempt to microzone (i.e., there was to be no attempt to locate actual faults on the regionalization maps, and variations of ground-shaking over short distances--on a scale of about 10 miles or less--were not to be considered). Such microzoning must be done by experts who are familiar with localized conditions, and there are many local jurisdictions that should undertake it, a point that is discussed further below.

Design Earthquake Ground Motion

The previous discussion referred to "design ground-shaking" without being specific as to the meaning of the phrase. Precise definition is difficult if not impossible but the concept is straightforward enough. The "design ground-shaking" for a location is the ground motion that an architect or engineer should have in mind when designing a building that is to provide protection for life safety.

At present, the best workable tool for describing the design ground-shaking is a smoothed elastic response spectrum for single degree-of-freedom systems (Newmark and Hall, 1969). 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 as a basic tool in the development of the regionalization maps and to include the effects of local ground conditions. In effect, the second policy decision was reinterpreted to mean for all locations roughly equal probability of exceeding at all structural periods the ordinates of the design elastic response spectrum for that location. Again, this statement should be looked upon as a general goal and not as one that can be strictly met on the basis of present knowledge.

This does not mean that a building must necessarily be designed for the forces implied by an elastic response spectrum. Later in this discussion it will be explained how, for purposes of the *Provisions*, elastic response spectra were converted into a formula for seismic design coefficient. For structures that can safely strain past their yield point, the forces determined in accordance with Sec. 4.2 are significantly smaller than those that would be determined from the corresponding elastic spectrum. However, the design engineer should keep the probable design ground motion in mind.

A smoothed elastic response spectrum is not necessarily the ideal means for describing the design ground-shaking. It might be better to use a set of four or more acceleration time histories whose average elastic response spectrum is similar to the design spectrum. This approach may be desirable for buildings of special importance but is not feasible for the vast majority of buildings. The use of a single time history generally is not adequate. This emphasizes that the design ground-shaking is not a single motion but rather a concept that encompasses a family of motions having the same overall intensity and frequency content but differing in some potentially important details of the time sequences of motions.

A significant deficiency of the response spectrum is that it does not by itself say anything about the duration of the shaking. To the extent that duration affects elastic response, it is accounted for by the spectrum. However, the major effect of duration is upon possible loss of strength once a structure yields. Although duration effects were not considered explicitly in drawing up the *Provisions*, in a general way it was envisioned that the design ground-shaking might have a duration of 20 to 30 seconds. The possibility that the design motion might be longer in highly seismic areas and shorter in less seismic areas was one of the considerations that influenced the assignment of Seismic Performance Categories in Sec. 1.4.

Ground Motion Parameters

In developing the design provisions, two parameters were used to characterize the intensity of design ground-shaking. These parameters are called the Effective Peak Acceleration (EPA), A_a, and the Effective Peak Velocity (EPV), A_v. These parameters do not at present have precise definitions in physical terms but their significance may be understood from the following paragraphs.

EPA and EPV can best be understood by considering them as normalizing factors for construction of smoothed elastic response spectra (Newmark and Hall, 1969) for ground motions of normal duration. The EPA is proportional to spectral ordinates for periods in the range of 0.1 to 0.5 second while the EPV is proportional to spectral ordinates at a period of about 1 second (McGuire, 1975). The ratio (for a 5 percent damped spectrum) of the spectral response ordinate at the appropriate period to the EPA or the EPV is set at a standard value of 2.5 in both cases.

For a specific actual ground motion of normal duration, EPA and EPV can be determined as illustrated in Figure C1-1. The 5 percent damped spectrum for the actual motion is drawn and fitted by straight lines at the periods mentioned above. The ordinates of the smoothed spectrum then are divided by 2.5 to obtain EPA and EPV. The EPA and EPV thus obtained are related to peak ground acceleration and peak ground velocity but are not necessarily the same as or even proportional to peak acceleration and velocity.

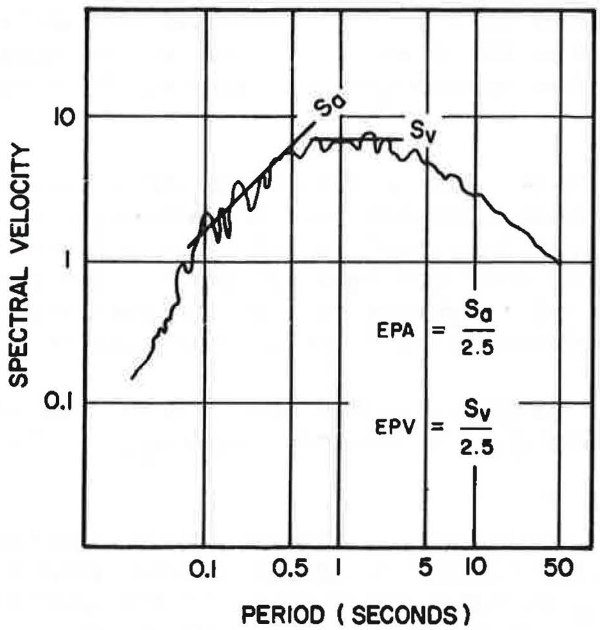


FIGURE C1-1
Schematic representation showing how EPA and EPV are obtained from a response spectrum.

When very high frequencies are present in the ground motion, the EPA may be significantly less than the peak acceleration. This is consistent with the observation that chopping off the highest peak in an acceleration time history has very little effect on the response spectrum computed from that motion, except at periods much shorter than those of interest in ordinary building practice. Furthermore, a rigid foundation tends to screen out very high frequencies in the free-field motion. On the other hand, the EPV generally will be greater than the peak velocity at large distances from a major earthquake (McGuire, 1975). Ground motions increase in duration and become more periodic with distance. These factors will tend to produce proportionally larger increases in that portion of the response spectrum represented by the EPV.

<u>*Sec. 1.4.1 (Ground Motion Parameters/Map for EPA)*</u>

If an earthquake is of very short or very long duration, it is necessary to correct the EPA and EPV values to more closely represent the event. It is well documented that two motions having different durations but similar response spectra cause different degrees of damage, the damage being less for the shorter duration. In particular, there have been numerous instances where motions with very large accelerations and short durations have caused very little or even no damage. Thus, when expressing the significance of a ground motion to design, it is appropriate to decrease the EPA and EPV obtained from the elastic spectrum for a motion of short duration. On the other hand, for a motion of very long duration, it would be appropriate to increase the EPA and EPV. There are at present, however, no agreed-upon procedures for determining the appropriate correction; it must be done by judgment.

Thus, the EPA and EPV for a motion may be either greater or smaller than the peak acceleration and velocity although the EPA generally will be smaller than peak acceleration while the EPV will be larger than the peak velocity. Despite the lack of precise definitions, the EPA and EPV are valuable tools for taking into consideration the important factors relating ground-shaking to the performance of a building.

At any specific location, either the EPA or the EPV may govern the design of a building. In general, however, it is desirable to know both values.

For purposes of computing the lateral force coefficient in Sec. 4.2, EPA and EPV are replaced by dimensionless coefficients, A_a and A_v respectively. A_a is numerically equal to EPA when EPA is expressed as a decimal fraction of the acceleration of gravity (e.g., if EPA = 0.2g, then A_a = 0.2). A_v is proportional to EPV as explained below in the discussion of "Implied Risk."

Map for EPA

The development of a map for EPA for the contiguous 48 states was facilitated by the work of Algermissen and Perkins (1976). Their map (Figure C1-2) is based on the principles of seismic risk (Cornell, 1968; Algermissen and Perkins, 1972).

Several steps are involved in the preparation of such a map:

- Source zones and faults, in which or along which significant earthquakes can occur, are identified and brought together on a source zone map.

- For each source zone or fault, the rate at which earthquakes of different magnitude can occur and the maximum credible magnitude are estimated.

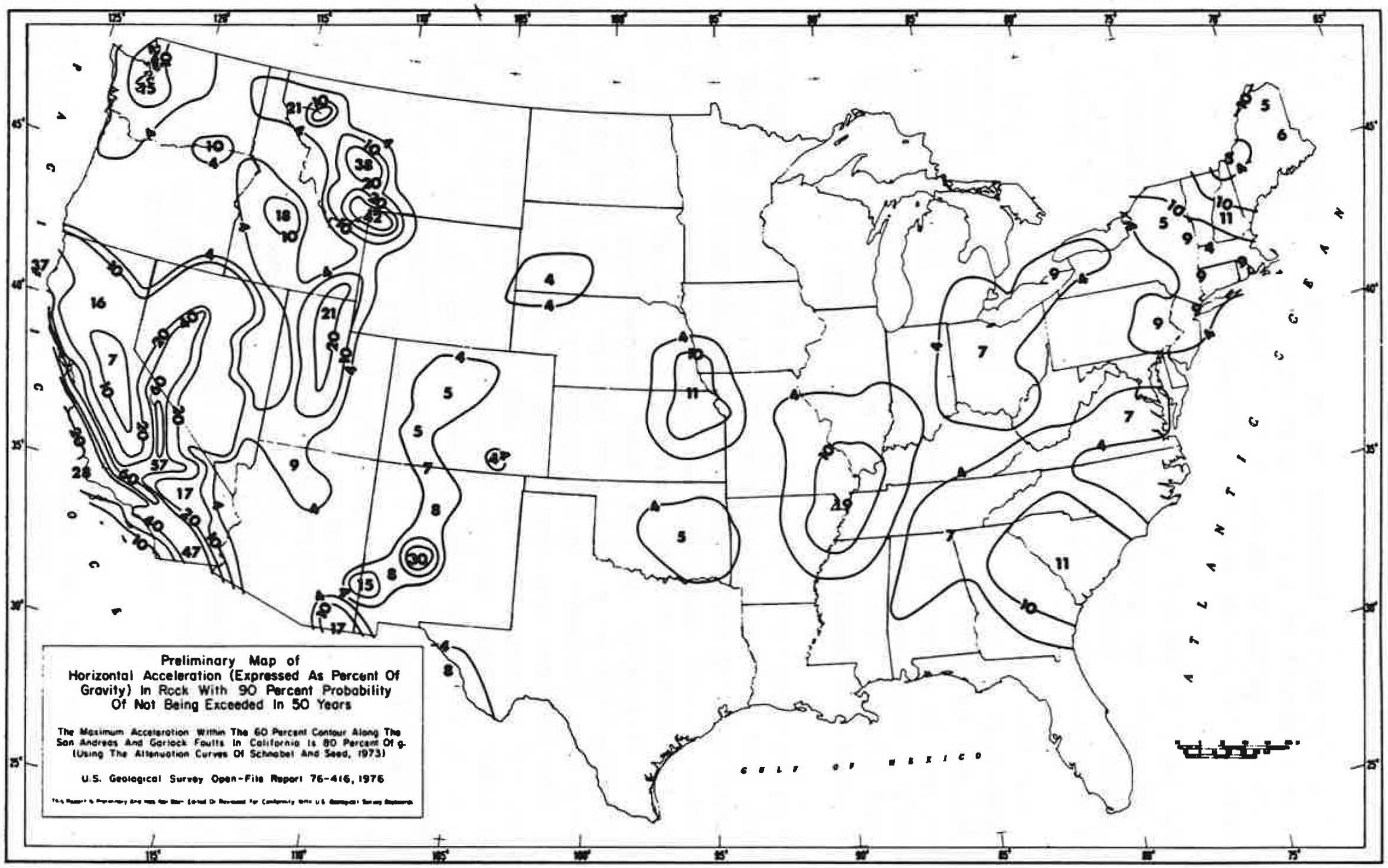


FIGURE C1-2

Seismic risk map developed by Algermissen and Perkins.

Sec. 1.4.1 (Map for EPA)

- Attenuation laws are used to give the intensity of shaking as a function of magnitude and distance from an epicenter.

- With the foregoing information as input, a computer program based on probabilistic principles can generate values that are then used to produce contours of locations with equal probabilities of receiving specific intensities of ground-shaking.

Algermissen and Perkins relied primarily on historical seismicity adjusted, where possible, by geological and tectonic information. The Algermissen-Perkins map shows contours of peak acceleration on rock that have a 10 percent probability of being exceeded in 50 years.

A contour map for EPA for the contiguous states was developed during the Applied Technology Council study (1978) that led to development of these provisions and is given in Figure C1-3. (This map was later converted into the map in Figure 1-1 of Chapter 1 by shifting contours to lie along county lines; see the discussion of county-by-county maps below.) It gives EPA for firm ground, which includes shale deposits of stiff cohesive soils and dense granular soils as well as rock.

The map of EPA is in many ways quite similar to the Algermissen-Perkins map and, indeed, was influenced by preliminary versions of that map. In adapting a map such as the Algermissen-Perkins map to the purposes of the *Provisions*, it was necessary to judge how acceleration as used in their study is related to EPA and how the "rock" of their study relates to the "firm ground" of the *NEHRP Recommended Provisions*. To produce a map appropriate as a basis for design it is desirable to use smoothed contours and, further, it is necessary to decide how to treat an area (e.g., New England and the Middle Atlantic states) where the accelerations in the Algermissen-Perkins map lie just below one of the arbitrarily selected contour levels. Seismologists from various parts of the country were asked to comment on proposed versions of the EPA map and suggested what were, in effect, alternate versions of the source areas. Also studied were other proposed maps--prepared from data in Culver et al. (1975) and published by Wiggins et al. (1977), Foss (1977), and others, using similar principles but different interpretations of historical seismicity and geological evidence. All of this evidence was taken into account where deemed appropriate by adjusting the locations of contours for EPA. Figure C1-3, having literally been drawn by a committee, lacks some of the internal consistency of the Algermissen-Perkins map but was judged to provide the best current estimate of the geographic variation of EPA for purposes of design.

Perhaps the most significant difference between Figures C1-2 and C1-3 occurs in the area of highest seismicity in California. Within this region, the Algermissen-Perkins map has contours of 0.6g. On the other hand, the map for EPA has no values higher than 0.4g. There are several reasons for this difference, all contributing to the decision to

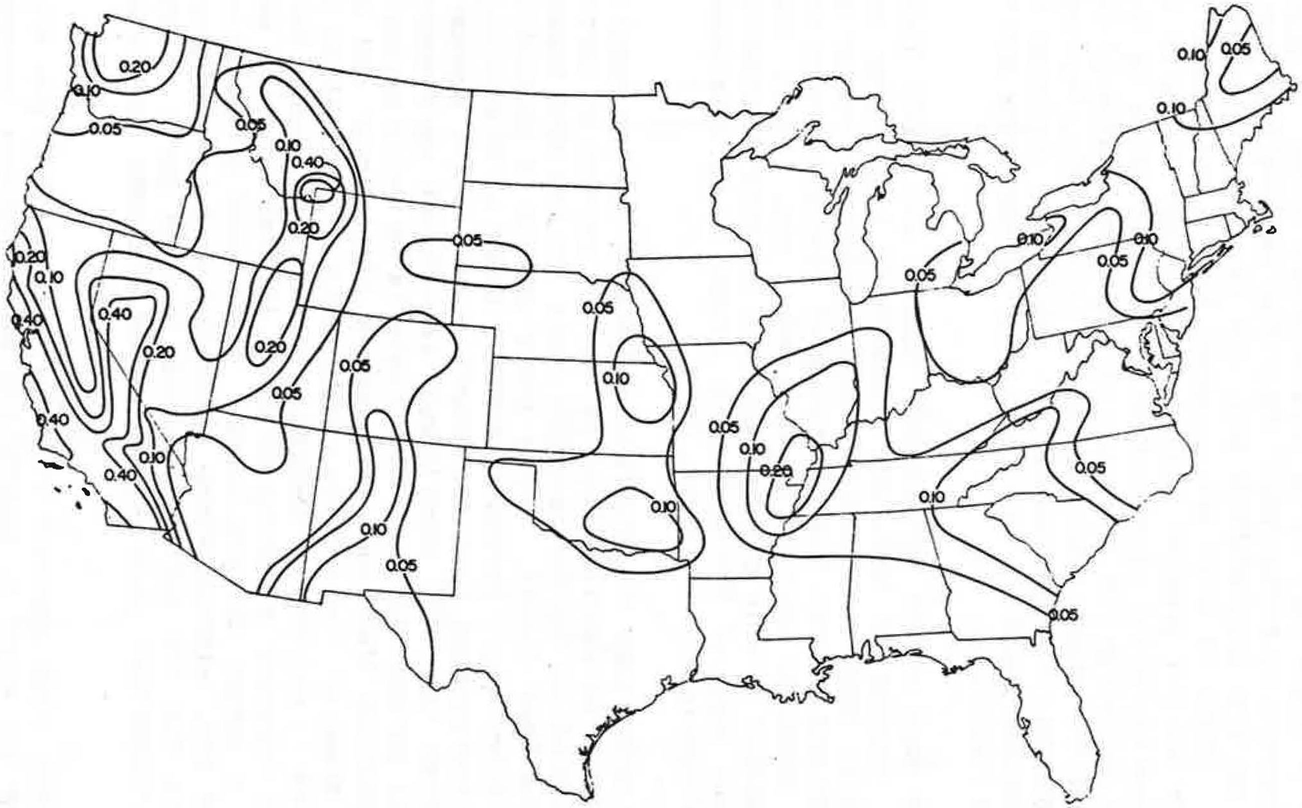


FIGURE C1-3

Contour map for effective peak acceleration (EPA) coefficient, A_a, for the continental United States.

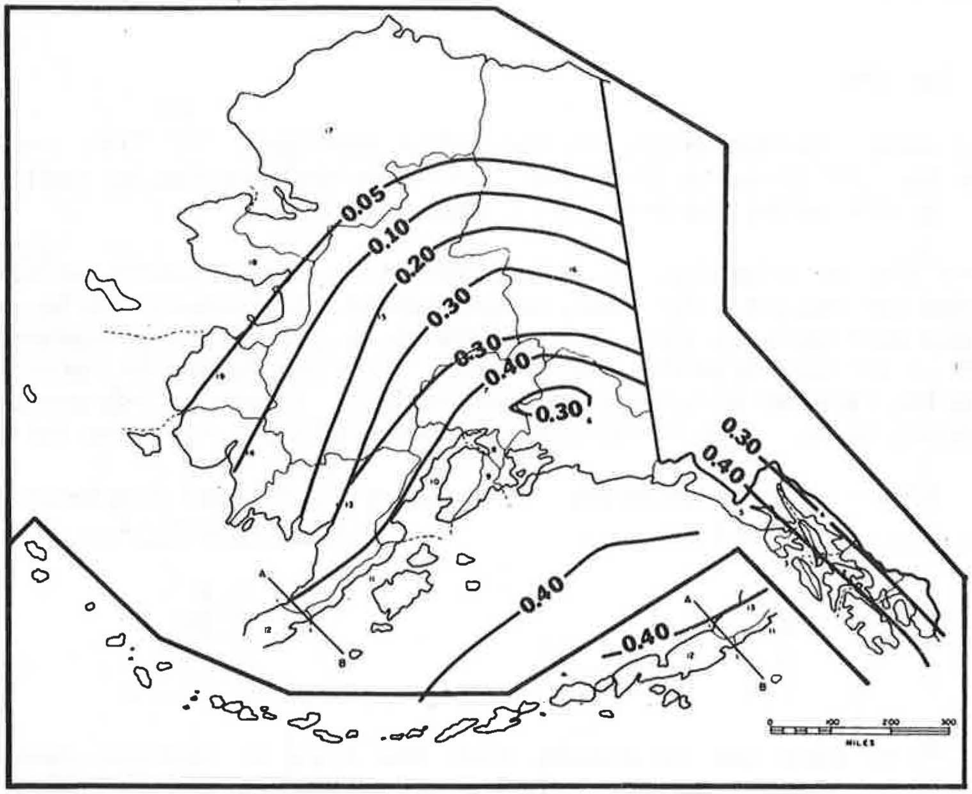
Note that the numbers on the contours are values of EPA in units of acceleration or gravity. They were used to prepare Figure 1-1 in Chapter 1 of the Provisions.

limit EPA to 0.4g. One factor is the basic difference between peak acceleration and EPA. There is doubt among many professionals that large earthquakes will cause very large accelerations except in quite localized spots influenced by topography. Many also believe that there is an upper limit to the acceleration that can be transmitted even through dense soil. There is also the argument that a building code requiring design for an EPA greater than 0.4g will not really bring about more earthquake-resistant construction. Finally, while by the formal logic used to establish EPA there may be locations inside of the 0.4g contour where higher values would be appropriate, contouring such small areas would amount to microzoning. In short, the decision to limit the EPA to 0.4g was based in part on scientific knowledge and in part on judgment and compromise.

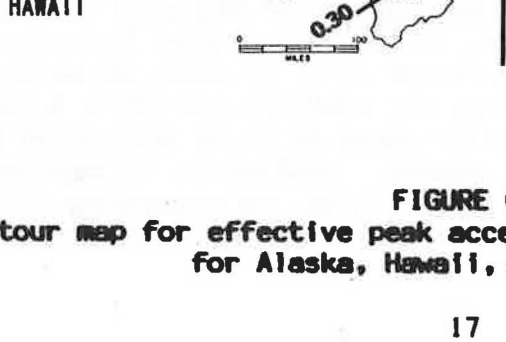
Figure C1-4 presents maps of EPA for Alaska, Hawaii, and Puerto Rico. In these areas, no studies of the type produced by Algermissen and Perkins were available; however, there had been conducted a number of seismological studies and seismic risk analyses in connection with the Alaskan pipeline, proposed nuclear power plants, etc. There also existed past and proposed seismic zoning maps. All of this information was used to construct maps of EPA that were judged to be consistent with the map for the contiguous 48 states.

It has already been noted that the Algermissen-Perkins map was heavily influenced by historical seismicity--that is, by the pattern of earthquakes that have occurred during the past 150 years (on the West Coast) to 350 years (on the East Coast). Where there was solid geological evidence that this rather short period of history might be misleading, this evidence was incorporated into the source model. This approach means that areas which have not experienced significant earthquakes during the historical period, and for which there is no solid geological basis for suspecting that such earthquakes might occur end up being designated as areas of low seismic risk. Careful examination of old earthquake records is necessary; however, some historic events felt in one location and recorded as being centered in that location may actually have been a larger distant event. These same difficulties apply to the map of EPA, although some geological and seismological studies did lead to the EPA being increased in some parts of the country where the historical record alone would indicate low seismicity.

Critics of the seismic risk approach rightfully argue that the historical record is far too short to justify the extrapolations inherent in the approach. Moreover, the most widely used procedures assume that large earthquakes occur randomly in time so that the fact that a large earthquake has just occurred in an area does not make it less likely that a large earthquake will occur next year. In light of current understanding of earthquake occurrences, this assumption is of limited



ALASKA



HAWAII

PUERTO RICO

FIGURE C1-4
Contour map for effective peak acceleration (EPA) coefficient, A_a, for Alaska, Hawaii, and Puerto Rico.

validity. However, at present there is no workable alternative approach to the construction of a seismic design regionalization map that comes close to meeting the goal of the second policy decision.

Map for EPV

No general mapping study is currently available for EPV. Hence, the maps for EPV (Figures C1-5 and C1-6) were constructed by modifying the map for EPA using the principles described below.

Since EPV is velocity, it is appropriately expressed in units such as inches per second. For ease in developing the formulas in Sec. 4.2, it proved desirable to also express EPV by a dimensionless parameter (A_v) that is an acceleration coefficient. This parameter is referred to as velocity-related acceleration coefficient. Figures C1-5 and C1-6 show contours of A_v. The relationship between EPV and A_v is as follows:

Effective Peak Velocity (in./sec)	Velocity-Related Acceleration Coefficient, A_v
12	0.4
6	0.2
3	0.1
1.5	0.05

The first step was to assume that the elastic response spectrum for firm ground would apply along the contours for EPA = 0.4g in Figure C1-3. The shape of this response spectrum, as described below, was obtained from analyses of actual strong motion records at distances of 20 to 50 miles from moderate to large earthquakes in California. If EPA = 0.4g, it is necessary to have EPV = 12 inches per second to construct this spectrum.

A similar assumption was made for all the peaks of the contour map for EPA--that is, at all locations where a contour gives the highest EPA in a region. For example, the EPV was set at 3 inches per second along the contour for EPA = 0.1g in the vicinity of the Appalachian Mountains and South Carolina.

A study by McGuire (1975) based on strong motion records in California has provided data concerning the attenuation of EPV with distance. For an earthquake of large magnitude, it was found that the distance required for EPV to decrease by a factor of 2 is about 80 miles. Thus, in the western part of the country, the contours for EPV = 6 inches per second were located at a distance of about 80 miles outside of the contours for EPV = 12 inches per second. Similarly, in Washington and Utah where the highest contour is at 0.2g, corresponding to EPV = 6 inches per second, the next contour for EPV = 3 inches per second was located about 80 miles away.

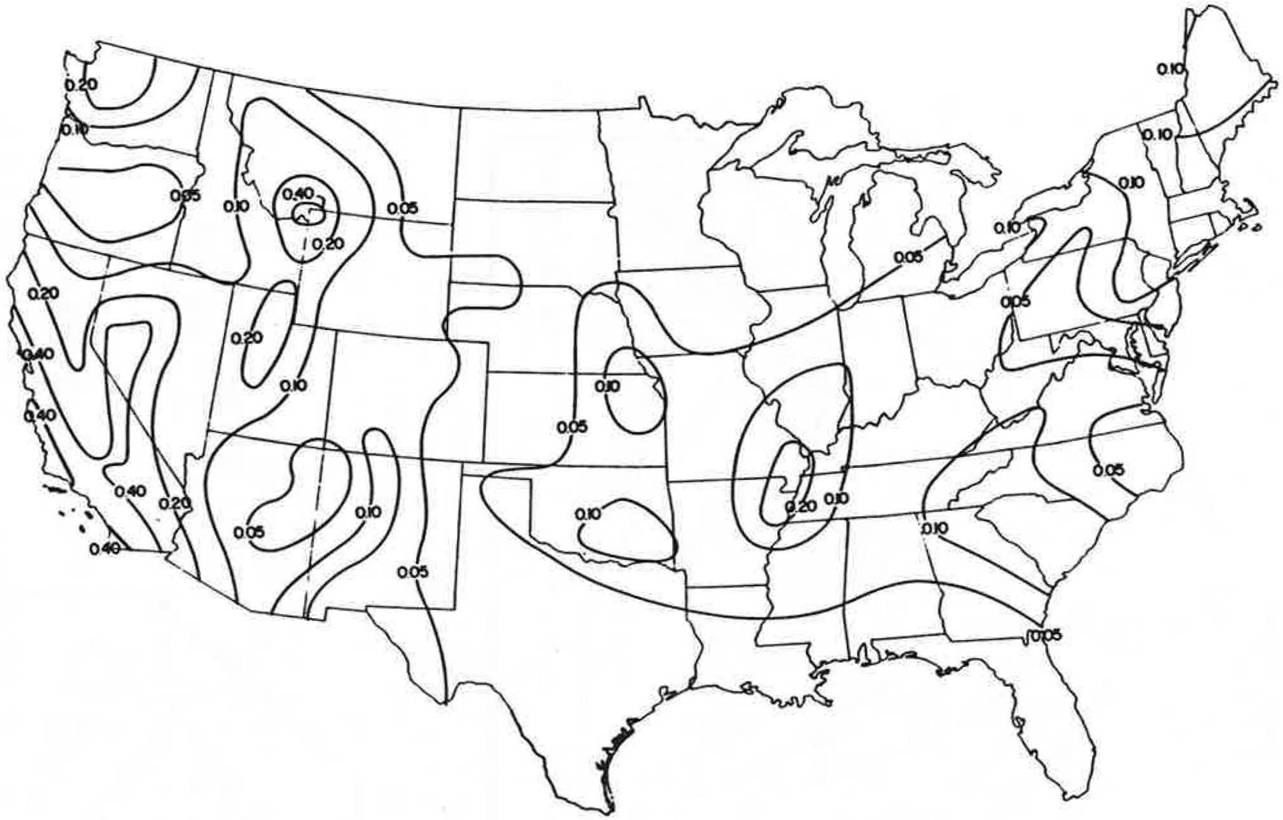


FIGURE C1-5

Contour map for effective peak velocity-related acceleration (EPV) coefficient, A_v, for the continental United States.

Note that the contours show values of A_v for use in Eq. C1-1.

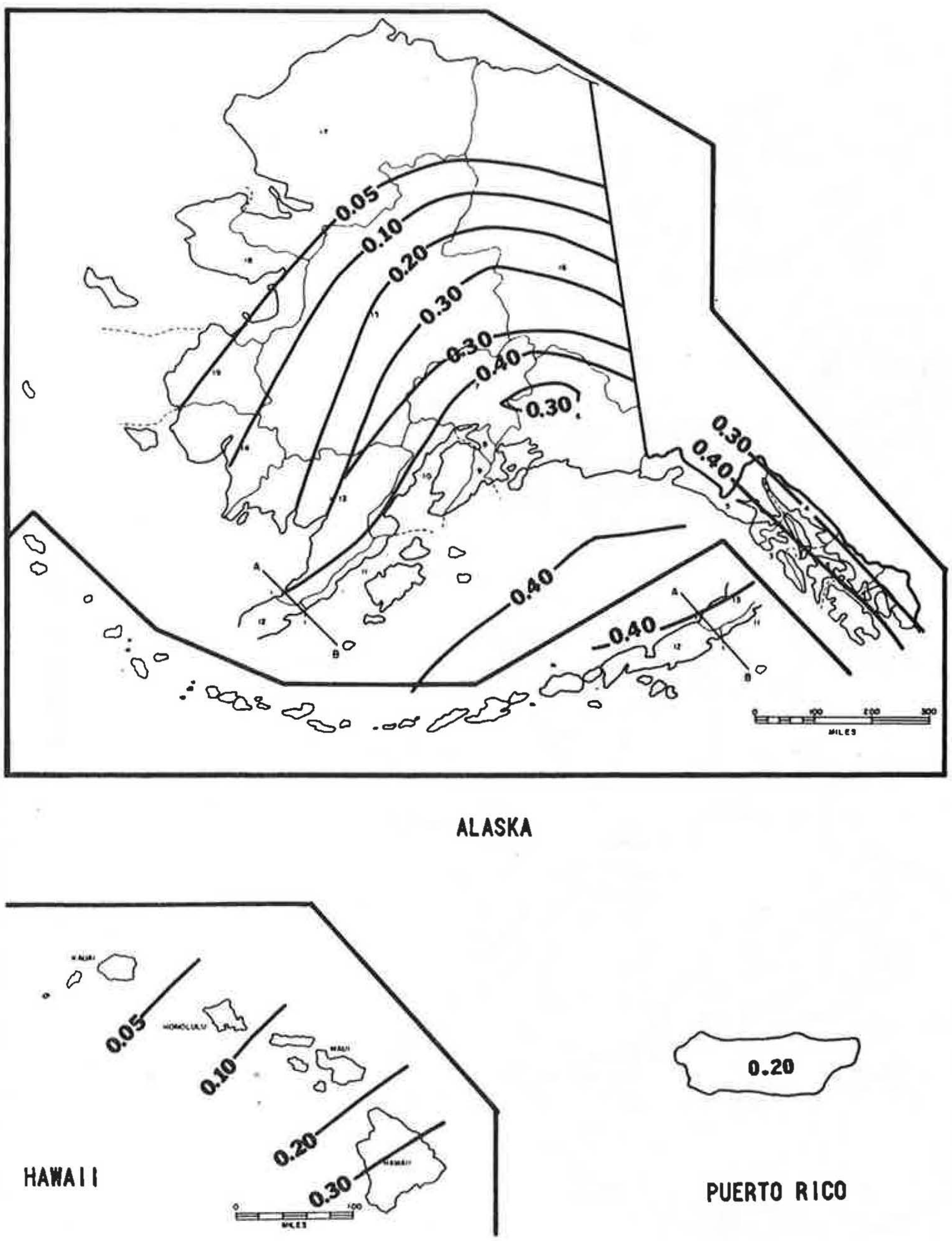


FIGURE C1-6
Contour map for effective peak velocity-related acceleration (EPV) coefficient, A_v, for Alaska, Hawaii, and Puerto Rico.

The strong-motion data available to McGuire were inadequate beyond a distance of about 100 miles. To estimate the attenuation of EPV beyond this distance, it was assumed that EPV at large distances from an earthquake is related to modified Mercalli intensity (MMI). It was further assumed that the logarithm of EPV would be linearly proportional to MMI. Data from large earthquakes in California suggested that MMI decreased roughly linearly with distance, which would translate into EPV continuing to halve at equal increments of distance. Thus, the contours subsequent to those located as described above also were spaced at about 80 miles.

For the Midwest and East, it was necessary to rely entirely on information about the attenuation of MMI (Bollinger, 1976). It appears that MMI decays logarithmically with distance and that for the first 100 miles from a large earthquake the attenuation in these regions is roughly the same as in the West. This would imply that the distance required for EPV to halve increases with distance. Thus, starting from the contour for EPV = 6 inches per second centered on southeastern Missouri, the contour for EPV = 3 inches per second would be about 80 miles away and the contour for EPV = 1.5 inches per second would be 160 miles beyond that for 3 inches per second.

In all cases, it was stipulated that a contour for EPV should never fall inside the corresponding contour for EPA. For example, the location of the contour for EPV = 3 inches per second in southcentral Illinois was determined by the contour for EPA = 0.1g rather than by distance from the contour for EPV = 6 inches per second.

After these various rules were applied to produce a set of contours for EPV, considerable smoothing was done and contours were joined where they fell close together. These steps were taken in light of the rather meager knowledge available about EPV at the time.

It would be highly desirable to have maps of EPV prepared using methods similar to those that have been used for peak acceleration. This was done for the northern half of California and gave results that are consistent with the contours on Figure C1-5. The maps in Figures C1-5 and C1-6 were deemed to be consistent with the state of the art.

Risk Associated with EPA and EPV

The probability that the recommended EPA and EPV at a given location will not be exceeded during a 50-year period is estimated to be about 90 percent. Given the present state of knowledge, this probability cannot be estimated precisely. Moreover, since the maps were adjusted and smoothed, the risk may not be the same at all locations. It is believed that this probability of not being exceeded is in the range of 80 to 90 percent. The use of a 50-year interval to characterize the probability is a rather arbitrary convenience and does not imply that all buildings are thought to have a useful life of 50 years.

It must be emphasized that the 90 percent probability of not being exceeded was not established initially as a criterion for selecting the EPA and EPV. A suitable level of EPA for the more seismic regions of California was selected on the basis of various considerations, some of which were mentioned above. Contours based on this level appeared to agree reasonably well with the level of acceleration determined by Algermissen and Perkins at the California border (California was not included in their earlier working maps) so their map was used as a guide for the rest of the country.

A probability of not being exceeded can be translated into other quantities such as mean recurrence interval and average annual risk. A 90 percent probability of not being exceeded in a 50-year interval is equivalent to a mean recurrence interval of 475 years or an average annual risk of 0.002 events per year. These other quantities have physical meaning only if averaged over very long periods of time--tens of thousands of years. In particular, a mean recurrence interval or return period of 475 years does not mean that the earthquake will occur once, twice, or even at all in 475 years. With present knowledge, there is no practical alternative to assuming that a large earthquake is equally likely to occur at any time, and quantities such as return period only indicate the likelihood that such an event will occur.

Figure C1-7, which is based on information supplied by Algermissen and Perkins from their study, indicates the probabilities of not being exceeded if other levels of EPA were to be selected. For example, consider a location on the contour for EPA = 0.2g in Figure C1-3. At this location, there is about a 60 percent probability that an EPA of 0.1g will not be exceeded during a 50-year interval. Similarly, there is 98 percent probability that the EPA will not exceed 0.35g. The dashed portions of the curves indicate possible extrapolations to larger and smaller annual risks. What this upper limit might be in any seismic area and especially in the less seismic areas is a matter of great debate; some experts feel that the upper limit is the same as for highly seismic areas although the probability of such an extreme EPA occurring is, of course, very, very small.

The probability that the ordinates of the design elastic response spectrum will not be exceeded at any period is approximately the same as the probability that the EPA and the EPV will not be exceeded. This is true because the uncertainty in the EPA and EPV that will occur in a future earthquake is much greater than the uncertainty in spectral ordinates, given the EPA and EPV. Thus, the probability that the ordinates of the design elastic response spectrum will not be exceeded during a 50-year interval is also roughly 90 percent, at least in the general range of 80 to 95 percent.

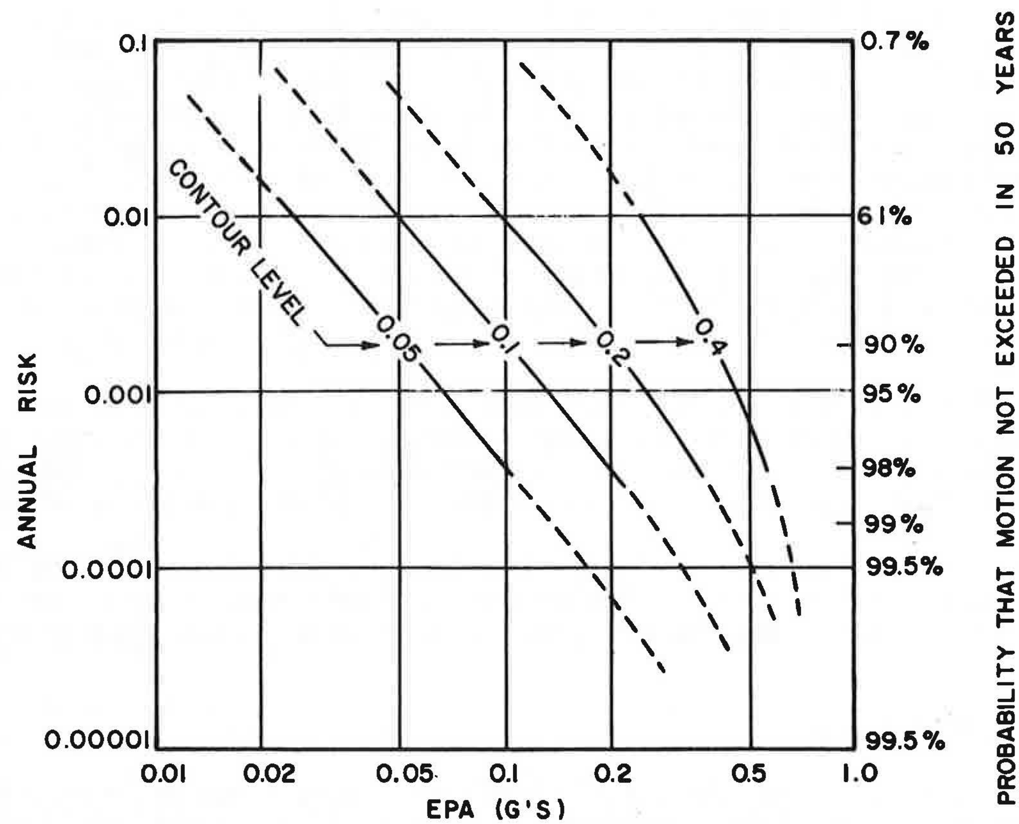


FIGURE C1-7
Annual risk of exceeding various EPAs for locations on the indicated contours of EPA in Figure C1-3.

Design Elastic Response Spectra

It is generally agreed that the characteristics of ground-shaking and the corresponding spectra are influenced by:

- The characteristics of the soil deposits underlying the proposed site,
- The magnitude of the earthquake producing the design ground motions,
- The source mechanism of the earthquake producing the ground motions, and
- The distance of the earthquake source from the proposed site and the nature of the travel path geology.

Although it is conceptually desirable to specifically consider all four factors, it is not now possible to do so because adequate data are lacking. Sufficient information is available to characterize in a general way the effects of specific soil conditions on EPA and spectral shapes. The effects of the other factors are so little understood at this time that they often are not considered in spectral studies. However, detailed spectral studies have shown that large portions of the response spectra can be closely represented using a scaling proportional to the EPA and EPV values (Blume et al., 1973, Newmark et al., 1973, Mohraz, 1976). The two maps can be easily used to represent the anticipated change in the shape of response spectra with the increase in distance from the seismic source zone by a direct adaptation of the response spectra for motions close to the seismic source zone.

The *Provisions*, therefore, only consider the effects of site conditions and the distance from the seismic source zone. At such times as the potential effects of other significant parameters can be delineated and quantified, the *Provisions* can be modified to reflect these effects.

Thus, the starting points in the development of the ground motion spectra are the seismic design regionalization maps that express by contours the EPA and the EPV that would be developed on firm ground.

Site Conditions

The fact that the effects of local soil conditions on ground motion characteristics should be considered in building design has long been recognized. Most countries considering these effects have developed different design criteria for several different soil conditions. Typically, these criteria use up to four different soil conditions. The ATC study (1978) that generated the preliminary version of the *Provisions* resulted in the use of three Soil Profile Types that were considered in the late 1970s to be different enough in seismic response to warrant separate seismic coefficients (s factors).

On the basis of the available body of data, the three conditions were selected as follows:

1. Soil Profile Type S_1

    a. Rock--of any characteristic whether it be shale-like or crystalline in nature. As a general rule, such material is characterized by a shear wave velocity greater than about 2,500 fps.

    b. Stiff soil conditions or firm ground--including any site where soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

2. Soil Profile Type S_2

   Deep cohesionless or stiff clay soil conditions--including sites where the soil depth exceeds about 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

3. Soil Profile Type S_3

   Soft-to-medium stiff clays or sands--characterized primarily by several tens of feet of soft-to-medium stiff clay with or without intervening layers of sand or other cohesionless soils.

Experience from the September 1985 Mexico City earthquake (see the discussion below), however, has prompted the addition of a fourth Soil Profile Type, S_4, for profiles with over 70 feet of soft clays or silts characterized by shear wave velocity of less than 400 fps.

Effective Peak Accelerations for Different Site Conditions

Based on the use of the four different site conditions grouped into the original three soil profiles outlined above, the values of EPA for rock conditions were modified to determine corresponding values of effective peak ground acceleration for the other site conditions. This modification was based on a statistical study of the peak accelerations developed at locations with different site conditions and the exercise of judgment in extrapolation beyond the data base.

After evaluating these effects and rounding out the results obtained, the values of EPA were further modified as follows: For the first three soil types--rock, shallow stiff soils, and deep cohesionless or stiff clay soils (Soil Profile Types S_1 and S_2)--there is no reduction. For the fourth soil type--soft to medium clays (Soil Profile Type S_3 and the new S_4)--a reduction factor of 0.8 is used. It should be pointed out that statistical data show that the reduction effect is not constant for all ground motion levels and the value of the reduction factor is generally smaller than is recommended here.

Spectral Shapes

Spectral shapes representative of the different soil conditions discussed above were selected on the basis of a statistical study of the spectral shapes developed on such soils close to the seismic source zone in past earthquakes (Seed et al., 1976a and 1976b; Hyashi et al., 1971).

Sec. 1.4.1 (Spectral Shapes)

The mean spectral shapes determined directly from the study by Seed et al. (1976a and 1976b), based on 104 records mostly from earthquakes in the western part of the United States, are shown in Figure C1-8. These spectral shapes also were compared with the studies of spectral shapes conducted by Newmark et al. (1973), Blume et al. (1973), and Mohraz (1976) and with studies for use in model building regulations. It was considered appropriate to simplify the form of the curves to a family of three by combining the spectra for rock and stiff soil conditions leading to the normalized spectral curves shown in Figure C1-9. The curves in this figure thus apply to the three soil conditions in the original provisions, and a line for the new Type S_4 has been added.

The four conditions corresponding to the four lines are described as follows:

- Soil Profile Type S_1--Rock of any characteristic, either shale-like or crystalline in nature (such material may be characterized by a shear wave velocity greater than 2,500 feet per second), or stiff soil conditions where the soil depth is less than 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

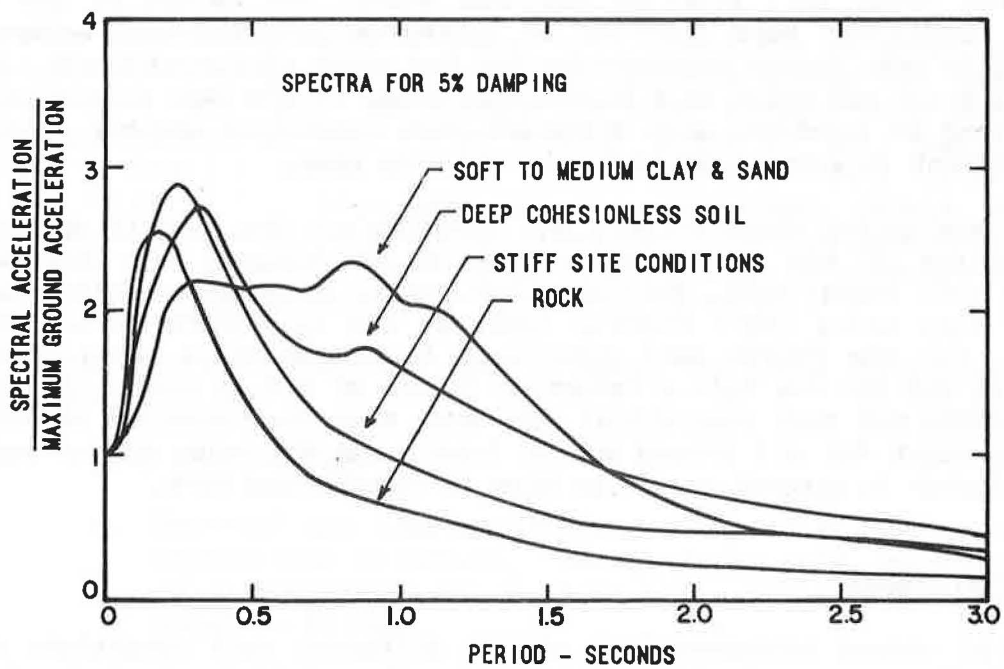


FIGURE C1-8
Average acceleration spectra for different site conditions (Seed et al. 1976a and 1976b).

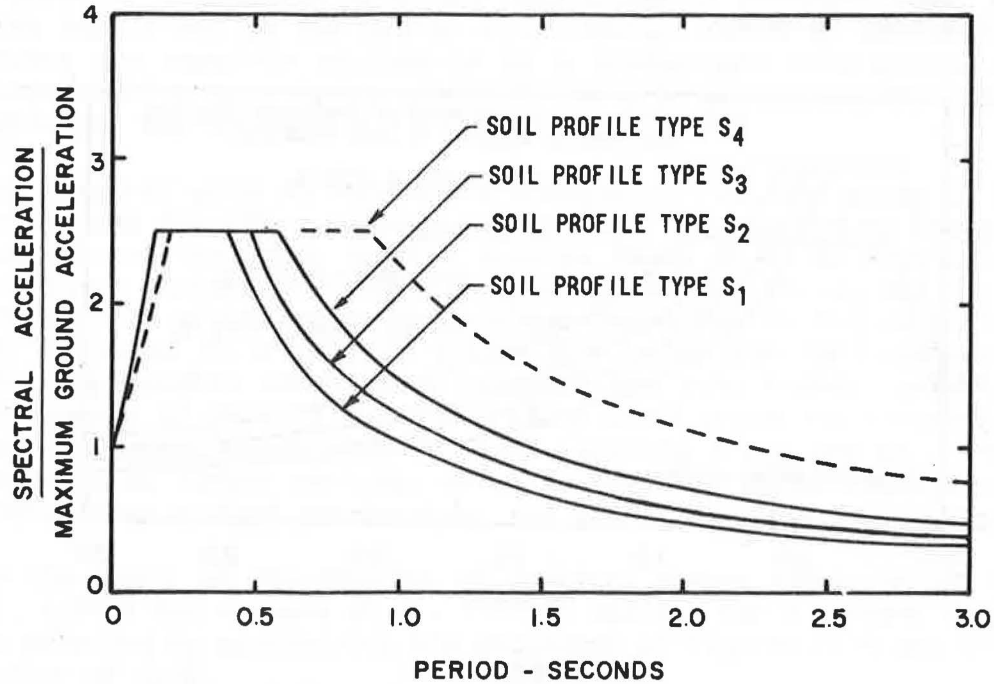


FIGURE C1-9
Normalized response spectra recommended for use in building codes.

- Soil Profile Type S_2--Deep cohesionless or stiff clay soil conditions, including sites where the soil depth exceeds 200 feet and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

- Soil Profile Type S_3--Soft-to-medium stiff clays and sands characterized by 30 feet or more of soft- to medium-stiff clay with or without intervening layers of sand or other cohesionless soils.

- Soil Profile Type S_4--Soft clays or silts greater than 70 feet in depth and characterized by a shear wave velocity of less than 400 feet per second.

Recommended ground motion spectra for 5 percent damping for the different map areas are thus obtained by multiplying the normalized spectra values shown in Figure C1-9 by the values of effective peak ground acceleration and the correction factor of 0.8 if Soil Profile Type S_3 or S_4 exists. The resulting ground motion spectra for Map Area 7 are shown in Figure C1-10. The spectra from Figure C1-10 are shown on Figure C1-11 plotted in tripartite form. It can be readily seen on Figure C1-11 that for all soil conditions the response spectra in the period range of about 1 second are horizontal or equivalent to a constant spectral velocity.

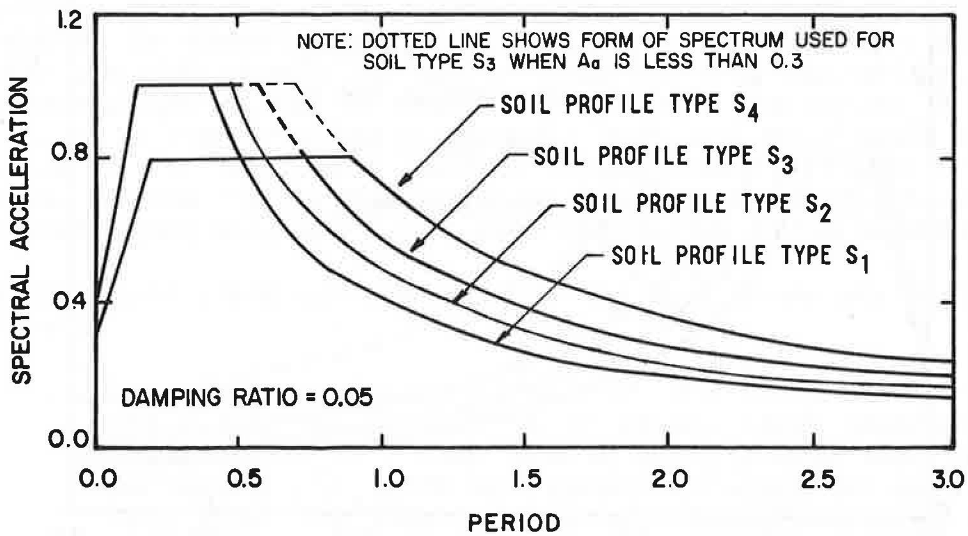


FIGURE C1-10
Ground motion spectra for Map Area 7 (A_a = 0.4).

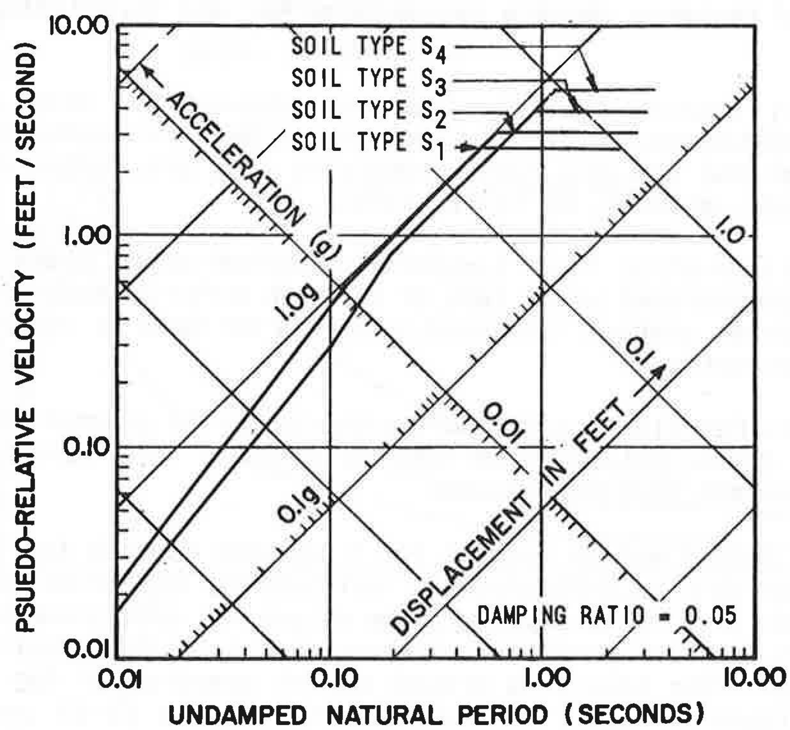


FIGURE C1-11
Ground motion spectra for Map Area 7 (A_a = 0.4).

The use of a simple soil factor to produce the different curves in Figure C1-10 produces a direct approximation of the effect of local site conditions on the design requirements. This direct method eliminates the need for estimation of a predominant site period and computation of a soil factor based on the site period and the fundamental period of the building.

The spectral velocity values are proportional to the values of A_v given on the map for EPV. For close-by motions represented by the innermost contours on the maps, spectra such as those shown on Figure C1-10 and C1-11 are applicable. Where the two contour values (A_a and A_v) differ, the portion of the response spectrum controlled by the velocity should be increased in proportion to the EPV value and the remainder of the response spectra extended to maintain the same overall spectral form. An example of this is shown on Figure C1-12 where the response spectra for Las Vegas and a site in South Carolina are compared. The higher response at longer periods, which is believed to be representative of motion from distant earthquakes, can be readily seen.

On the basis of the studies of spectral shapes conducted by Blume et al. (1973) and Newmark et al. (1973), spectra for 2 percent damping may be obtained by multiplying the ordinates of Figures C1-9 and C1-10 by a factor of 1.25.

Spectra for vertical motions may be determined with sufficient accuracy by multiplying the ordinates of the spectra for horizontal motions by a factor of 0.67.

Mexico City 1985 Experience

In September 1985, Mexico City was shaken by a great earthquake that was centered some 400 km to the southwest. The shaking experienced in Mexico City during this event varied markedly depending on the subsurface soils conditions. This shaking was most intense within a region underlain by an ancient dry lake bed composed of soft clay deposits. The recorded motion was very long (nearly 2 minutes). Furthermore, a significant segment of these long-duration motions exhibited nearly harmonic motions with a period of about 2 seconds. As a result, the spectral amplitudes of the lake bed motions in this period range were very large and, in fact, were much greater (by factors of 15 to 20) than the corresponding spectral amplitudes from rock site motions recorded at comparable epicentral distance.

The most significant damage induced by this earthquake was confined to the lake bed region and occurred in 5- to 15-story buildings with small-strain natural periods of about 0.8 to 1.0 second. As the elements of such buildings began to crack and yield during shaking, the period of the buildings began to lengthen. As this lengthening period approached the 2-second period of the dominant ground motions, the structural response progressively intensified as it entered into resonance with the underlying soil response. Further, the long duration of the nearly harmonic ground-shaking provided ample time for this resonance condition to develop and caused the buildings to undergo many cycles of intense shaking. This, in turn, led to a progressive increase in building damage and, in many cases, to eventual collapse.

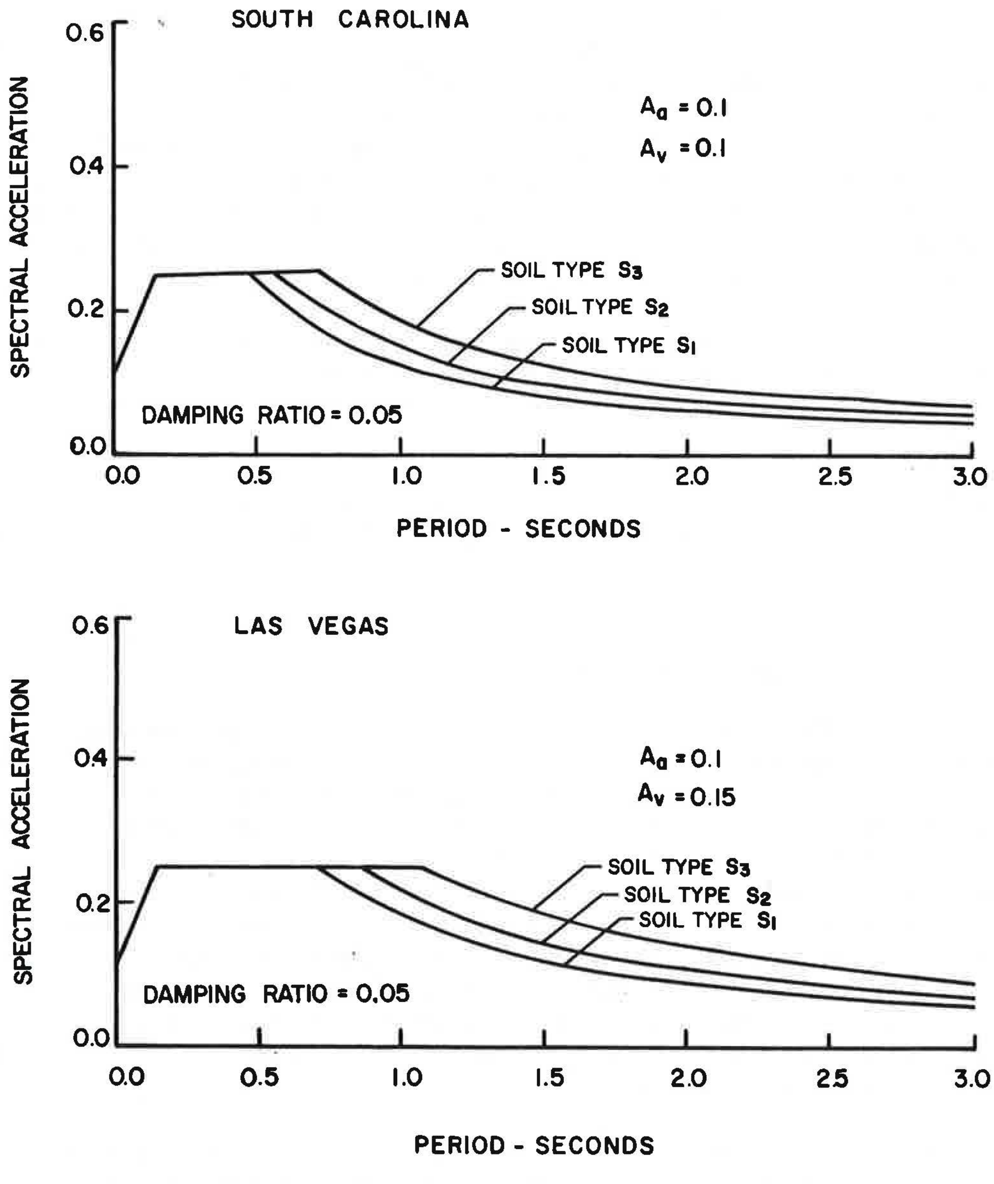


FIGURE C1-12
Examples showing variation of ground motion spectra in different tectonic regions.

Because of the similarity of the soft clay deposits in Mexico with those of certain Soil Profile Type S_4 areas in California, there is concern about the possible occurrence of extensive damage to structures on such deposits due to resonance-type response characteristics similar to those described above. Therefore, special dynamic analysis procedures have been adopted for certain classes of structures judged to be susceptible to such damage. The purpose here is to define the conditions for which these special analysis procedures are to be applied.

From the evaluation of the observed damage in Mexico City it is judged that damage to structures on Type S_4 soils is unlikely if the natural period of the structure is short compared to that of the site. Therefore, structures with natural periods of less than 0.5 seconds have been excluded as candidates for these special analysis procedures. In addition, the occurrence of the resonance-type response due to progressive lengthening of the structural period was judged to be dependent on the ratio of this period, T, to the characteristic site period, T_s, as determined from a site response analysis. Based on the Mexico City experience, it has been determined that this resonance-type response is most likely for buildings having calculated periods equal to or greater than 0.7 second and that dynamic analysis is required for such buildings to properly evaluate the related effects of S_4 soil conditions for the longer period buildings.

A 0.7-second calculated building period is comparable to the calculated predominant soil period for S_4 soils considered as a single layer system using the formula:

$$T_{soil} = 4H/V_s$$

with H = 70 feet and V_s = 400 feet per second.

Lateral Design Force Coefficients

The equivalent lateral force method of design requires that a horizontal force be accommodated in the structural design. The magnitude of this force is a function of several parameters including the map area, the type of site soil profile, the fundamental period of the building, and the type of building construction.

In a design provision or code, it is distinctly advantageous to express the lateral design force coefficient in as simple a manner as possible. The recommended procedure for determining the lateral design force coefficient C_s is given in Sec. 4.2 as follows:

$$C_s = 1.2\ A_v S/RT^{2/3}. \qquad \text{(C1-1)}$$

Sec. 1.4.1 (Lateral Design Force Coefficients)

The value of C_s need not exceed 2.5 A_a/R. For Type S_3 soils when A_a is equal to or greater than 0.3, the value of C_s need not exceed 2 A_a/R. The soil profile coefficient S is given in Table 3-1 as follows:

Type	S Factor
S_1	1.0
S_2	1.2
S_3	1.5
S_4	2.0

Curves for these relations are plotted in Figure C1-13. The curves are not precisely the same as the spectral shapes in earlier figures. The procedure by which these curves were derived for the response spectra curves is as follows: As buildings become larger and more complex there arise, in addition to the increase in modes of vibration, many modes by which severe damage can be initiated. There is also a greater likelihood that high ductility requirements may be concentrated in a few stories of the building. These factors, when combined with the importance of larger buildings to the community, suggest that the larger and longer period structures should be given a more conservative criteria or weighting factor. It was judged that this weighting factor should make the lateral force coefficient approximately 50 percent greater at a period of 2 seconds for the stiff soil condition than would be obtained by direct use of the response spectrum. This increase should gradually reduce as the building period shortens.

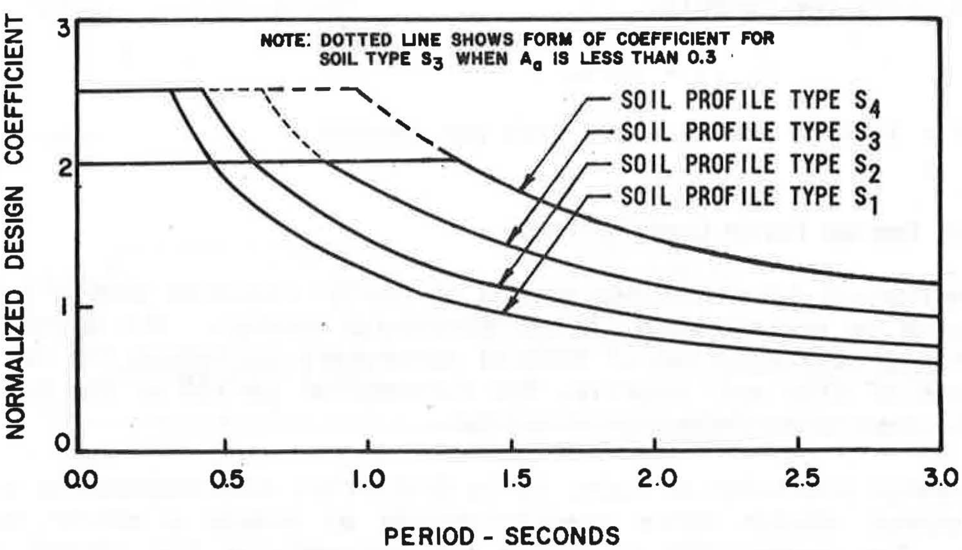


FIGURE C1-13
Normalized lateral design force coefficients ($A_a = A_v = 1.0$).

A comparison between the lateral design force coefficients and the free-field ground motion spectra is shown on Figure C1-14.

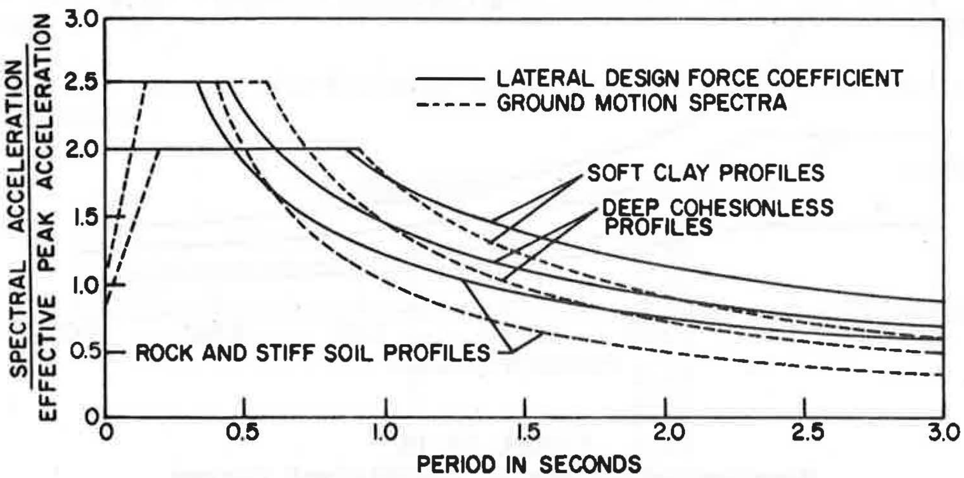


FIGURE C1-14
Comparison of free-field ground motion spectra and lateral design force coefficients.

In the application of these recommendations the values of A_a and A_v may not be equal so that the lateral force coefficient curves will be different from those discussed above. To illustrate the varying effects obtained from the use of the lateral force equation, the respective curves of C_sR for shallow stiff soil sites for several cities are shown on Figure C1-15.

County-by-County Maps

It generally is recognized that the exposure to seismic hazard decreases as the distance from an active seismic region increases. It was in recognition of this simple premise that abandonment of the broad uniform zoning then being considered was one of the first recommendations made during the ATC project leading to the preliminary version of the *Provisions*. This recommendation suggested that seismic zoning should be on the basis of the contours shown on Figures C1-3 through C1-6 with interpolation being used to obtain values between the contour levels. It soon became apparent, however, that interpolation by the user might produce some difficulties in coastal areas and along the international borders (where interpolation would require extension of the contours beyond national boundaries). These difficulties, combined with the problem of defining a simple interpolation procedure with no ambiguity, led to an alternate method of producing zoning maps--the use of Map Areas with specified values of A_a or A_v with boundaries along

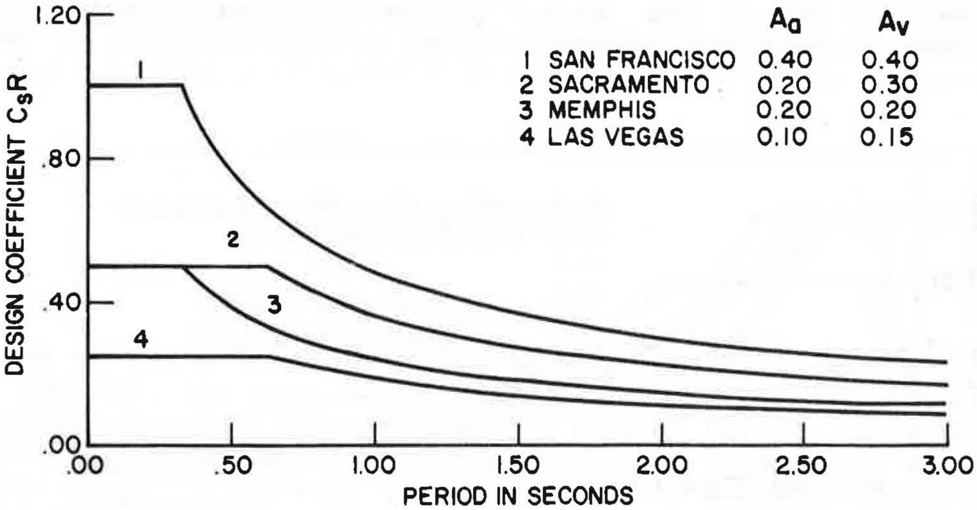


FIGURE C1-15
Representative design coefficient curves for Soil Type S_1 in four different locations.

those of political jurisdictions. The simplest form of subdividing the contiguous states was to use county boundaries. This decision was reviewed and eventually reversed by the BSSC primarily because the county zone procedure is particularly cumbersome in the West. However, because both county-by-county maps and contour maps are available, both continue to be published. Future maps (including those in the "Appendix to Chapter 1") will be in contour form printed over a county line backgound.

Either the county-by-county seismic design regionalization maps presented in Chapter 1 of the *Provisions* as Figures 1-1 and 1-2 or the contour maps in Figures 1-3 and 1-4 are used to determine the A_a and A_v coefficient values, respectively. The county-by-county maps were prepared by assuming that each county should be represented by the highest contour in that county. In developing the county-by-county map, intermediate contours were drawn for coefficient values of 0.3 and 0.15, which are listed in Table 1-1 but are not shown on Figures C1-3 and C1-5. It can be seen that the procedure of assigning the same value throughout a county produces discontinuities in some areas of the map. As indicated above, it is strongly recommended that local jurisdictions with better definition of the earthquake hazard consider microzonation of those counties that are at discontinuities on the county-by-county maps.

Seismicity Index

A Seismicity Index was included in the 1985 Edition of the *Provisions*. This Seismicity Index, which was simply a grouping of similar values of A_v, has been replaced in the 1988 Edition by citation of explicit values of A_v.

The values of the coefficients A_a or A_v associated with Map Areas are as follows:

Map Area	A_a or A_v
7	0.40
6	0.30
5	0.20
4	0.15
3	0.10
2	0.05
1	0.05

Note that A_a and A_v are not necessarily the same for a given location because the location may be in different Map Areas on the two maps.

Building Cost Implications

Determining the effect of the *Provisions* on the initial cost of buildings is enormously complex and it is possible to arrive at many different answers depending upon:

- The role in society of the person answering the cost question,
- Whether or not the building is required to remain functional after a major earthquake, and
- Whether or not some seismic design requirements already apply to the building.

For new construction that need not remain functional following an earthquake, the change in cost as a result of seismic design can vary enormously from project to project. The major factors influencing the cost of complying with the *Provisions* are:

- The complexity of the shape and structural framing system for the building. (It is much easier to provide seismic resistance in a building with a simple shape and framing plan.)
- The cost of the structural system (plus other items subject to special seismic design requirements) in relation to the total cost of the building. (In many buildings, the cost of pro-

viding the structural system may be only 25 percent of the total cost of the project.)

- The stage in design at which the provision of seismic resistance is first considered. (The cost can be inflated greatly if no attention is given to seismic resistance until after the configuration of the building, the structural framing plan, and the materials of construction have already been chosen.)

The approximate cost impacts resulting from implementation of an earlier version of the *NEHRP Recommended Provisions* were determined by Weber (1985) during a BSSC study of the societal implications of using improved seismic design provisions. Weber's study was based on the results of 52 case studies that compared the costs of constructing the structural components of a wide variety of buildings designed according to two distinct criteria: the prevailing local building code and a proposed set of improved seismic safety provisions (as noted above, an earlier version of the *NEHRP Recommended Provisions*). Some of the case studies also compared the structural engineering design time required for the two design criteria. The case studies included multifamily residential, office, industrial, and commercial building designs in nine cities that cover the range of seismic hazard levels found in the United States (Los Angeles, Seattle, Memphis, Pnoenix, New York, Chicago, Ft. Worth, Charleston, and St. Louis).

These case studies were developed on the basis of the BSSC trial design program conducted in 1983-84. This program, which is described in detail in Appendix C (The BSSC Program on Improved Seismic Safety Provisions) of this *Commentary* volume, was established to evaluate the usability, technical validity, and cost impact of the application of a somewhat amended version the 1978 ATC provisions. It is important to note that these provisions were further refined as a result of the trial design program, during the BSSC balloting of the 1985 Edition, and again during the updating process resulting in the 1988 Edition. Thus, as noted by the BSSC (1984b): "Some buildings showing high cost impacts [would] be significantly affected by new amendments...that should tend to reduce the impact."

During the trial design program, 17 professional design organizations from the 9 cities were retained to prepare trial designs of the following building types:

1. Low-, mid-, and high-rise residential (R) buildings,
2. Mid- and high-rise office (O) buildings,
3. One-story industrial (I) buildings, and
4. Two-story commercial (C) buildings.

Each of the following structural systems was included:

1. Lateral load systems
   a. Shear walls
      (1) Cast-in-place concrete
      (2) Precast and prestressed-precast concrete
      (3) Masonry
      (4) Plywood on wood studs
   b. Braced frames--conventional steel
   c. Unbraced frames
      (1) Cast-in-place concrete both special and ordinary (as defined in the amended *Tentative Provisions*)
      (2) Steel, both special and ordinary, conventional and pre-engineered
2. Vertical load systems
   a. Bearing wall buildings
      (1) Walls
         (a) Cast-in-place concrete
         (b) Precast and prestressed-precast concrete
         (c) Masonry
         (d) Plywood on wood studs
      (2) Floors
         (a) Concrete slabs, both cast-in-place and precast, ordinary and prestressed
         (b) Steel joists with decks and slabs
         (c) Wood framing with plywood decks and lightweight concrete fill
   b. Framed buildings
      (1) Cast-in-place concrete flat slabs, waffle slabs, pan joists, and beam and slab systems, both ordinary and prestressed
      (2) Precast concrete, both ordinary and prestressed
      (3) Steel girder and purlin, beam and joist, and long-span truss systems with decks and slabs
      (4) Wood framing

As noted above, each building was designed twice: once according to an earlier version of these *Provisions* and once according to the prevailing local code for the particular location of the design. Basic structural designs (complete enough to assess the cost of the structural portion of the building), partial structural designs (special studies to test specific parameters, provisions, or objectives), and partial nonstructural designs (complete enough to assess the cost of the nonstructural portion of the building) were prepared and design and construction cost estimates were developed.

Weber's cost impact data based on the results of the trial designs are presented below in summary form. In presenting these data, Weber distinguished between two separate cases: (1) the communities that were not using a seismic code of any kind (e.g., Memphis and St. Louis) and (2) the communities that were using a seismic code (e.g., Charleston and Seattle).

Sec. 1.4.1 (Building Cost Implications)

According to Weber, the construction cost impact of the earlier version of these *Provisions* generally depends on two major groups of factors:

- Those related to characteristics of the building itself and including such things as the planned occupancy of the building, the structural system used to support the building, the general shape of the building in terms of number of stories and floor plan, and the total size of the building.

- Those related to the location in which the building is to be constructed and including such things as the seismic hazard of the building site and the degree to which that hazard is reflected in the current local building code.

Table C1-1 presents an overview of the construction cost impacts by type of building occupancy. The third column in Table C1-1 presents the percentage change in construction costs for the structural components of the building, with the Local Code Design as the base, as estimated by the BSSC trial design engineering firms. As can be seen, the average change for the structural costs is 5.6 percent, with by far the largest change (11.2 percent) reported for the high-rise residential designs. This high average for residential buildings is significantly influenced by the extremely high estimates (46, 20, 17, and 16 percent) reported for four building designs.

The fourth column of Table C1-1 presents the projected percentage change in total building construction costs for each building occupancy type. These total cost changes were projected from the structural cost percentage changes by using data on structural cost as a percentage share of total building cost for each building occupancy type. The percentage shares are based on data from McGraw-Hill's *Dodge Construction System Costs* (1984), which reports the structural percentage share of total building cost for a large number of typical building designs. The shares for three of these typical building designs were averaged for each of the building occupancy types to derive the percentage shares used in Tables C1-1 and C1-2 and reported in the footnotes to the tables. The average projected change in the total construction cost over all 52 of the trial designs is 1.6 percent. The high-rise residential building designs have the highest total building cost impact with 3.3 percent, both because of the four designs with excessive costs mentioned above and the relatively high structural percentage share used for this type of building (30.0 percent).

Table C1-2 presents data similar to that in Table C1-1 but for each city grouped according to whether the city currently had a seismic building code or not. As expected, the average estimated change in the

TABLE C1-1
Percentage Changes in Structural Cost and Total Building Cost for the Trial Designs by Building Occupancy Type

Building Occupancy	Number of Designs	Estimated Change in Structural Cost (%)[a]	Projected Change in Total Cost (%)[b]
Low-rise residential[c]	9	3.6	0.7
High-rise residential[d]	12	11.2	3.3
Office	21	4.7	1.3
Industrial	7	1.5	0.5
Commercial	3	5.6	1.7
Average Percentage Change		5.6	1.6

[a]Percentage change in structural construction cost from the local code to early version of the *Provisions*, as estimated by the BSSC trial design engineering firms, 1983-1984.

[b]Projected percentage change in total building construction cost from the local code to early version of the *Provisions*, derived from estimated structural cost changes by using the following McGraw-Hill's, *Dodge Construction Systems Cost* (1984) data on structural cost as a percent of total building cost: low-rise residential, 18.1%; high-rise residential, 30.0%; office, 28.1%; industrial, 33.7%; commercial, 29.5%.

[c]Five or fewer stories.

[d]More than five stories.

TABLE C1-2
Percentage Changes in Structural Cost and Total Building Cost for the Trial Designs by City and City Group With and Without Seismic Provisions in Current Local Codes

City	Number of Designs	Estimated Change in Structural Cost (%)[a]	Estimated Change in Total Project Cost(%)[b]
Cities Without Seismic Provisions			
Chicago	10	2.5	0.7
Ft. Worth	3	6.1	1.5
Memphis	6	18.9	5.2
New York	7	7.3	2.1
St. Louis	3	4.5	1.3
Average Percentage Change		7.6	2.1
Cities With Seismic Provisions			
Charleston	3	-2.5	-0.6
Los Angeles	10	4.2	1.3
Phoenix	6	6.9	1.9
Seattle	4	-1.1	-0.3
Average Percentage Change		3.1	0.9
Overall Average Percentage Change		5.6	1.6

[a]Percentage change in structural construction cost from the local code to early version of the *Provisions*, as estimated by the BSSC trial design engineering firms, 1983-1984.

[b]Projected percentage change in total building construction cost from the local code to early version of the *Provisions*, derived from estimated structural cost changes by using the following McGraw-Hill's, *Dodge Construction Systems Cost* (1984) data on structural cost as a percent of total building cost: low-rise residential,18.1%; high-rise residential, 30.0%; office, 28.1%; industrial, 33.7%; commercial, 29.5%.

structural cost is considerably higher (more than twice as high) for those cities with no seismic provisions in their local codes than for those with seismic provisions: 7.6 percent versus 3.1 percent. A similar relationship holds for the projected change in total building cost: 2.1 percent for cities without seismic provisions versus 0.9 percent for those already having some seismic provisions in their local codes.

The estimates made by the trial design firms of the change in structural design time that was expected to be required once the firms become familiar with the provisions were divided into the following categories:

1. Negligible change,

2. Positive but unspecified change,

3. Positive specified change, and

4. Negative specified change (meaning that the newer provisions, once adopted and familiar to the design firms, would require fewer design hours than do the current codes).

Twenty-eight of the trial designs fell into the "negligible change" category. Eleven fell into each of the next two categories, and two fell into the fourth category.

In summary, Weber's study of the results of the BSSC trial design program provides some idea of the approximate cost impacts expected from implementation of the *NEHRP Recommended Provisions*. For the 29 trial designs conducted in the 5 cities (Chicago, Ft. Worth, Memphis, New York, and St. Louis) whose local building codes had no seismic design provisions, the average projected increase in total building construction costs was estimated to be 2.1 percent. For the 23 trial designs conducted in the 4 cities (Charleston, Los Angeles, Phoenix, and Seattle) whose local codes had seismic design provisions, the average projected increase in total building construction costs was estimated to be 0.9 percent. The average increase in costs for all 9 cities was estimated to be 1.6 percent. Although analyses of the cost effect of the 1985 and 1988 Editions of the *NEHRP Recommended Provisions* have not been conducted, it is anticipated that the modifications made to the earlier version studied would have little effect on cities subject to high seismic risk but would reduce the cost effects on cities subject to less risk.

The costs cited above obviously are of greatest interest to the owners of a proposed building. There are, however, other potential cost implications, each of which reflects the viewpoint of a different group in society.

Sec. 1.4.1 (Building Cost Implications/Implied Risk)

Any change in design requirements can potentially effect the suppliers of building materials and of proprietary building systems. In the short run, these changes may adversely affect the competitive advantage of an organization or industry. In the long run, however, American industry has always shown remarkable adaptability to new building regulatory requirements.

Adoption of new design requirements also may result in additional costs for the agency charged with administration and enforcement of the requirements. Such agencies are in a position similar to that of an engineering firm in that efforts for plan review and inspection may have to increase.

Implied Risk

The following discussion addresses methods for evaluating implied risk and presents one estimate of the risk implied by the *Provisions*. The word "risk" is used here in a general sense to indicate losses that may occur in the future at uncertain times and in uncertain amounts as a result of earthquake ground-shaking.

It is not possible by means of a building code to provide a guarantee that buildings will not fail in some way that will endanger people as a result of an earthquake. It may not be desirable for a code to attempt to ensure the absolute safety of buildings since the resources to construct buildings are limited. Society must decide how it will allocate the available resources among the various ways in which it desires to protect life safety. One way or another, the anticipated benefits of various life-protecting programs must be weighed against the cost of implementing such programs.

One reason a code cannot ensure absolute safety is the present (and probably future) inability to describe on a firm scientific basis the strongest earthquake ground-shaking that might possibly occur at any specified location. As long as this is the case, it is impossible to design for zero risk and, hence, a decision to design a building for a specified capacity has associated with it an ***implicit risk***. This risk may be quite small (e.g., 1 chance in 10,000 that a building will fail during an earthquake), but it is greater than zero.

None of the methods or estimates presented here are precise; indeed, they are quite crude and uncertain. They do, however, serve two very valuable purposes:

- They show the factors and considerations that influence overall risk and

- They give a general indication of the level of safety provided by the *Provisions* in comparison with other risks faced by society.

Expressing Losses

In general, losses may be in the form of damage and repair costs, injuries and fatalities, and the indirect adverse effects upon a community, region, or country. Because the emphasis of the *NEHRP Recommended Provisions* is on life safety, this discussion is specifically concerned with losses directly related to life safety. In many ways it might be more appropriate to use injuries and fatalities (i.e., "major casualties") as a measure of the risk to life safety. However, since many find it difficult to talk in terms of predicted major casualties and it is difficult to make accurate predictions concerning major casualties, this discussion will make use of an indirect measure of the risk to life safety--the risk of building failure where such failure would imply a threat to life safety. More precise definitions of failure will be discussed subsequently.

Expressing Probability

The time when the next major earthquake will affect a particular city is unknown as is the magnitude of that earthquake. The future losses sustained in that city may result from several moderate-sized earthquakes or from a single large earthquake. Since there is little agreement as to the specific nature of the most intense ground-shaking that might occur, especially in the less seismically active parts of the country, it is difficult to be specific about the largest possible losses that might occur. These considerations mean that the future losses are uncertain and some measure of probability must be used in the examination of such losses. This might be done in several ways, but two approaches are commonly used.

One way is the use of *average annual losses*. Risk might be expressed as the average dollar loss per year, the average major casualties per year, the average number of building failures per year, etc. Losses expressed in this way are *annual risks*. However, large earthquakes are very rare events, and losses averaged for such infrequent events may not give a meaningful portrayal of the large loss that might occur for one such event.

The second way is to define a threshold of loss and to estimate the probability that the threshold will be equaled or exceeded during some earthquake. For example, one might speak of the probability that the dollar cost of damage and repair will exceed $1 billion dollars during at least one earthquake during the next 50 years. The threshold might alternatively be some number of human casualties or some number of building failures.

General Procedure for Estimating Probability of Failure

The design earthquake ground motion by itself does not determine risk; the risk is also affected by the design rules, analysis procedures, and construction practices used in connection with the design ground motion. Thus, the overall risk to a building is determined by both the seismic hazard and the probable building performance. It is expressed by the following equation giving the average number of failures, f, per year for an individual building.

$$f = p[F|a] \frac{d\gamma}{da} da, \qquad \text{(C1-2)}$$

where

a = EPA or EPV as appropriate,

$P[F|a]$ = probability of failure if an intensity of shaking with EPA = a occurs, and

γ = annual rate at which intensities of shaking are exceeded (see Figure C1-7).

The integration is over all possible values of a. The average annual rate of failures can then be converted to the probability that failure will occur during some period of time. This is the same as the conversion between the left-hand and right-hand scales of Figure C1-7.

Estimated Performance of Buildings Designed According to the *Provisions*

The following paragraphs give rough estimates, based on experience and judgment, of the probability of failure occurring when a building designed in accordance with the *Provisions* is subjected to different levels of ground-shaking. However rough, the estimates should suffice for general guidance as to the degree of safety implicit in the *Provisions*. The estimates are intended to apply to a building of moderate size and complexity meeting the minimum requirements of the *Provisions*.

If the design ground motion were to occur, structural collapse--meaning collapse of part or, in extreme cases, of all of a building--should not be expected in buildings designed in accordance with the *Provisions*. (Failures due to design or construction errors cannot be prevented by design requirements alone; detailed design reviews and mandatory construction inspection are also necessary.) If a ground motion twice as strong as the design ground motion were to occur, there might be structural collapses in about 1 to 2 percent of the buildings designed in accordance with the *Provisions*. If a ground motion is three times as strong as the design earthquake motion, this percentage might be 5 to 10 percent.

If the design ground motion were to occur, there might be life-threatening damage in 1 to 2 percent of buildings designed in accordance with the *Provisions*. (In each building so damaged, on the average, about 1 percent of the occupants might be major casualties.) If ground motions two or three times as strong as the design ground motion were to occur, the percentage of buildings with life-threatening damage might rise to about 10 to 50 percent, respectively.

These estimates are presented in graphic form in Figure C1-16 to illustrate the expected performance of buildings designed for different EPAs. Possible extrapolations of the relations are suggested. The extrapolation toward low conditional probabilities of failure is difficult to estimate; in effect, one is asking what is the probability of major design and construction errors such that the building might "fail" during a very small ground motion.

Implicit Risk for a Single Building Versus a Group of Buildings

The information contained in Figures C1-7 and C1-16 has been used as input to Eq. C1-2 to compute failure probabilities for four buildings: one located on the contour in Figure C1-3 for 0.4g and designed for that EPA, one on the contour for 0.2g and designed for that EPA, and likewise for buildings located on the 0.10g and 0.05g contours. In each case, several different assumptions were made as to how the solid line in Figures C1-7 and C1-16 should be extrapolated.

It was found that, because of compensating trends, the probabilities of failure were roughly the same for each of the buildings. For buildings on the contours for 0.05g and 0.10g, the result is influenced strongly by the way in which the curves of Figures C1-7 and C1-16 are extrapolated to larger values of EPA or EPV. On the other hand, the results for a building located on the contour for 0.4g are influenced strongly by the extrapolations to smaller values of EPA or EPV.

Table C1-3 gives estimates for the probability that the two types of failure will not occur within a 50-year period. Note that these probabilities are more favorable than those for the design EPA or EPV. This simply means that a building generally will not fail just because the shaking in some earthquakes slightly exceeds the design EPA.

It must be emphasized that these estimates are very crude. All of the potential difficulties discussed in relation to estimating EPA apply even more strongly here.

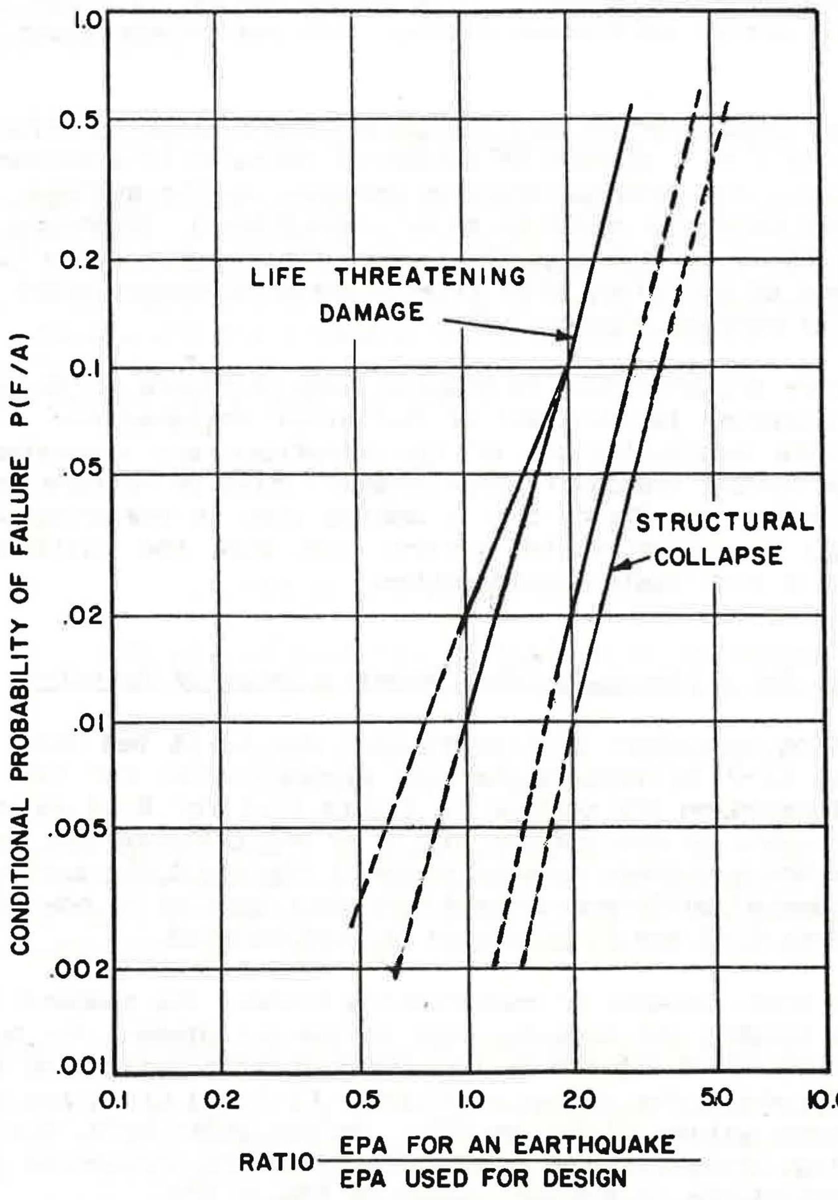


FIGURE C1-16
Probability of failure as a function of actual earthquake relative to design earthquake.

If there are a number of similar buildings at some location such that all buildings experience approximately the same shaking during any one earthquake, the probability that at least one of the buildings will fail is greater than the probability that any one particular building will fail. Calculations also have been made for this case assuming 100 similar buildings. Results are included in Table C1-3. This case represents, in a very crude way, the expected performance in any one city of new construction designed and constructed in accordance with the *Provisions*.

TABLE C1-3
Probability of Not Having Any Failures During a 50-Year Period (in percent)

	Type of Failure: Life-Threatening Damage	Type of Failure: Structural Collapse
Single building	99	99 to 99.9
100 buildings - 1 city	90	95
100 buildings - 5 cities	65	85

When one considers a series of cities, the probability that at least one failure will occur becomes even greater. To illustrate this, assume five cities each having 100 buildings designed in accordance with the *Provisions*. From Table C1-3 it is seen that the probability of a failure occurring is no longer insignificant.

These results emphasize that the perception of the level of safety achieved by the *Provisions* is different for the owner of a single building, the public officials of a city, and the public officials of a state.

Acceptable Risk

There are no laws in the United States that state an "acceptable number" of fatalities per person exposed per year or any other proposed definition of acceptable risk. There also are no judicial decisions that give firm guidance. Legislative bodies have chosen alternatives with implied risks that have been stated quantitatively. For example, in arriving at new seismic requirements for existing buildings, the Long Beach City Council opted for an alternative to which a risk of 10^{-6} fatalities per person exposed per year had been attached (the other alternatives implied smaller risks). Obviously there have been many other cases where legislative, judicial, and executive bodies have made choices that imply some level of risk. However, all such instances taken together do not constitute a firm set of precedents.

Sec. 1.4.1 (Acceptable Risk)

There have been attempts to determine an acceptable level of risk on fundamental grounds. For example, Wiggins (1975) compiled data for the risk in situations (driving, flying commercial airlines, accidents in the home) where people more or less knowingly exposed themselves to risk. These so-called voluntary risks are of the order of 200 fatalities per million people exposed per year. Wiggins then referred to the work of Starr (1969), who concluded that the public wants involuntary risks (such as from earthquakes) to be much smaller (say 100 to 10,000 times smaller) than voluntary risks. Thus, the acceptable risk from earthquakes might be between 1 and 0.01 fatalities per million people exposed per year.

As a second example, Figures C1-17 and C1-18 summarize data for the probability of man-made and natural disasters causing greater than various numbers of fatalities. Obviously, these data reflect past practice and not necessarily levels of risk that are desirable. If the "total man-caused" and "total natural" curves are reduced by 1,000 (so as to give a level of risk that would not contribute significantly to total overall risk) for a 50-year period, there would be a 2.5 percent probability of one or more such events.

The analysis provided above in the discussion of implied risk can be used, in a crude way, to provide risk estimates for comparison with Figures C1-17 and C1-18. Consider buildings of moderate size housing several hundred people, such that a structural collapse would--considering that buildings are usually unoccupied or lightly occupied for much of a week--on the average cause 100 fatalities. For the case of five cities with 100 buildings in each city, the frequency of an earthquake causing about 100 fatalities was estimated to be 0.003 events per year. With 50 cities with 100 such buildings each, the rate rises to 0.03 events per year. To the extent that this calculation is valid, it might then be concluded that the *Provisions* are not unduly conservative.

Another approach to determining an appropriate level of risk is by a cost-benefit analysis. Such analyses are difficult when lives are at stake but can be applied to the prospective loss aspect of earthquake damage. Although the *Provisions* have been written to minimize the hazard to life safety, as a by-product they will reduce damage costs--especially during moderate-sized earthquakes. In highly seismic areas where moderate earthquakes occur frequently, any increase in building costs will be offset by reduced costs of damage. In less seismic areas, however, seismic design requirements can be justified only in terms of life safety since the expected savings in damage during very infrequent earthquakes are not great enough to justify an average 1 percent increase in building costs.

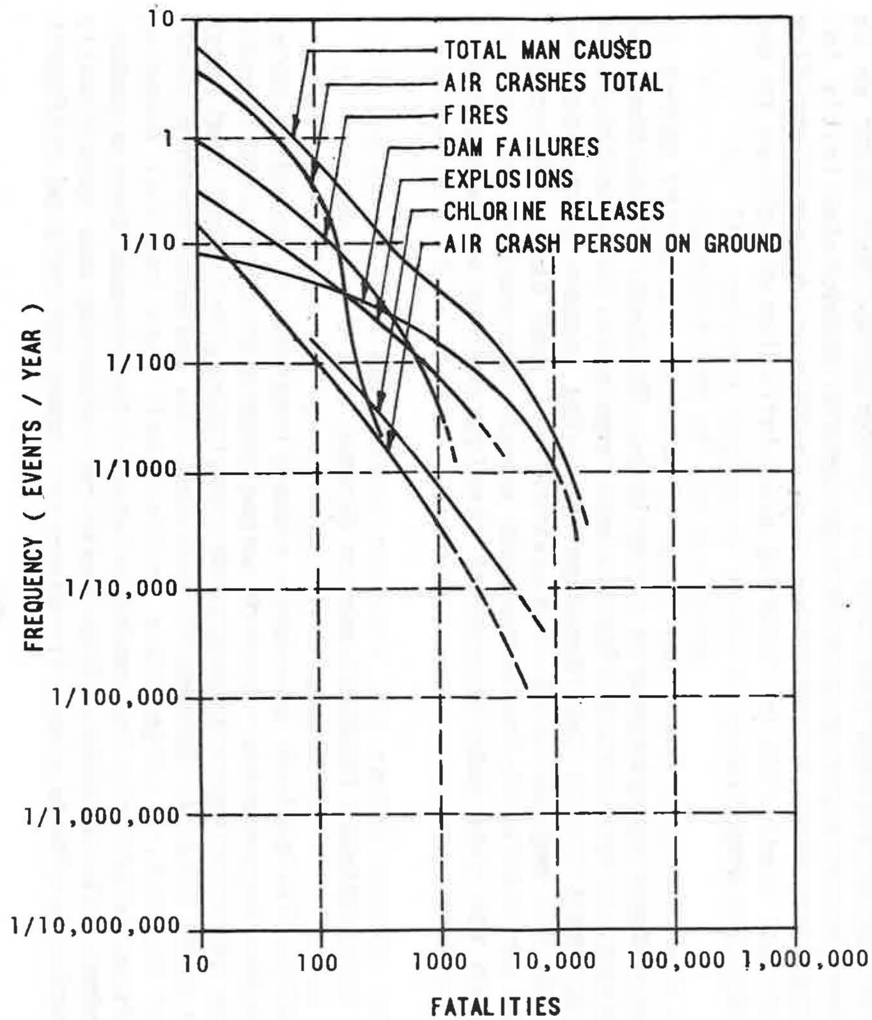


FIGURE C1-17
Fatalities due to man-caused failures
(U.S. Nuclear Regulatory Commission, 1976).

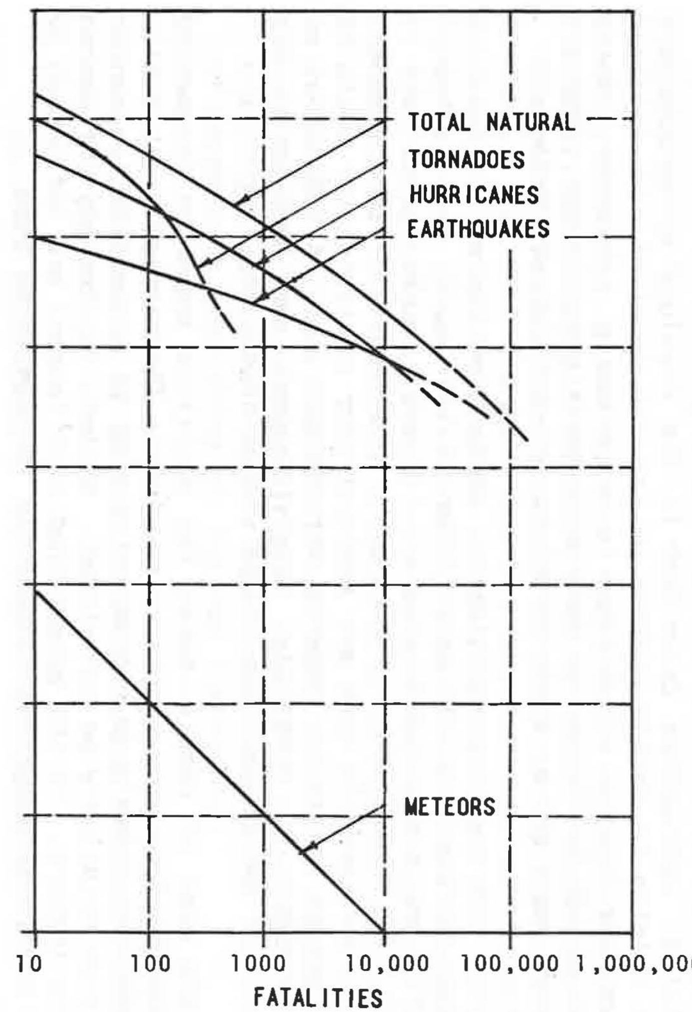


FIGURE C1-18
Fatalities due to natural disasters
(U.S. Nuclear Regulatory Commission, 1976).

Other Viewpoints

The technical approaches described in the previous paragraphs are useful in helping to decide whether or not the level of risk implicit in a proposed course of action is acceptable. However, these approaches do not by themselves make such decisions. Rather, they are made through legislative, administrative, and judicial processes.

In proposing and enacting legislation, administrative and legislative bodies have increasingly expressed interest in results from technical cost-benefit and risk-benefit studies. However, such bodies make it clear that they do not wish to be bound by the results of such studies, and it is understandable that any administrator or legislator would be very hesitant to explicitly endorse any non-zero risk of fatalities as being acceptable. Ultimately, administrators and legislators are guided by their own perceptions of the wishes of society.

Society--the mass of people--makes its decisions based on fragmented information and from many varying viewpoints. The people, individually and collectively, simply do not perceive risk in a quantitative manner that can even relatively be correlated. Society is strongly influenced by credible leaders. To the extent that such leaders are influenced by technical analyses, society is indirectly influenced by them.

Administrative bodies have the task of interpreting legislation so as to know how to apply it, and the act of interpretation implicitly involves decisions about acceptable risk. In this role, administrative bodies evaluate their risk by relating administrative directives to the ultimate in peer practice.

Often the courts become the final judge of whether a proposed course of action for mitigating a hazard is acceptable. The body of law that has been developed in the area of flood plain regulation is a useful guide to judicial reactions to hazard mitigation. The lesson is to match the severity of the regulation to the severity of the risk. The courts follow the principle of the reasonable person who strives to achieve this balance and uses data to support findings of the appropriate balance.

1.4.2 Seismic Hazard Exposure Groups

Historically, the typical occupancy classifications in building codes are based on the potential hazards associated with fire. Review and evaluation of existing building code provisions indicated that most occupancy type classifications do not meet the purpose of this document. For example, a large-scale enclosed-mall-type regional shopping complex is a relatively new architectural form representing a potentially high risk occupancy that existing codes do not specifically address properly. These classifications are based not only on different considerations than those related to seismic resistance but also, in some cases, on considerations that are contrary to good seismic performance.

Attention was given to the Model Code Standardization Committee's (MCSC) Code Change Proposal III-75-1, which recommended a series of change of occupancy designations to refer to the same use in all model codes. The MCSC changes, however, did not seem sufficiently varied to cover all issues related to seismic safety since they were limited to only seven broad fire-oriented classifications: assembly, business (including offices, factories, mercantile, and storage), educational, hazardous, institutional, miscellaneous structures, and residential.

A new approach was needed for defining occupancy exposure to seismic hazards based on a commonality of conditions proposed for the use of a building facility or space. These conditions would involve evaluation of parameters consisting of, but not limited to:

1. The number, age, and condition of the persons normally expected to be within or without the immediate environs of the building.

2. The size, height, and area of the building.

3. The spacing of the buildings relative to public rights-of-way over which the designer has no control relative to the future number of persons exposed to risk by the buildings.

4. The varying degree of built-in or brought-in hazards based on possible use of the building.

Accordingly, as development of the *Provisions* was beginning, occupancy types were regrouped and expanded to cover a complete range of factors critical to seismic safety in terms of life loss. The expanded classification types were derived from the 1973 *Uniform Building Code* (*UBC*) and are presented in Table C8-5, "Tentative Matrix," in the Chapter 8 Commentary. (Note that they were developed only for study purposes and are not intended as recommended changes to any building code.)

In terms of post-earthquake recovery and redevelopment, certain types of occupancies are vital to public needs. These special occupancies were identified and given specific recognition. In terms of disaster preparedness, fire and police stations, hospitals, and regional communication centers identified as critical emergency services should not be included in the same classification as retail stores, office buildings, and factories as is presently the case in some codes.

Because of vital public needs immediately following a natural disaster, attention was given to the preservation of strategic contents in distinct building types. For example, should storage facilities for medical supplies, critical foodstuffs, and other emergency materials require a higher seismic performance than the storage of less vital reserves and provisions? It was noted that disaster recovery officials initially considered the identification and protection of critical stocks needed during or immediately following an earthquake to be of paramount importance. This was not to imply that all warehouses and storage facilities must be designed for the ultimate protection of any or all contents. What was indicated was that warehouse facilities should be designed on the basis of their maximum level of intended function or, to state it another way, medical supply warehouses being designed under higher standards may house anything while storage facilities of lesser ratings may not store critical supplies unless brought up to a higher level of seismic performance.

Subsequent discussions with disaster recovery officials revealed that emergency contingency plans contemplated bringing needed medical and other recovery items including foodstuffs into a disaster area from outside staging areas. Therefore, no separate category of warehousing was required for the storage of critical materials. Table C8-3 thus has 9 occupancy groups, A through I, with some individual occupancies and groups bearing little or no relationship to current code groupings.

The occupancies then were consolidated into five basic groups by making a few compromises. This consolidation was done in an effort to place those occupancies initially listed in the "Tentative Matrix" into groups that shared common component performance criteria. The consolidation indicated that these groups were easily identifiable by use patterns, confirmation of the original occupancy-component-performance criteria rating. This intermediate group was:

<u>Group I</u>--fire, police, hospitals.

<u>Group II</u>--public assembly, open air stands, day care, schools, colleges, retail stores, shopping centers, offices, hotels, apartments, emergency vehicles, power utilities.

<u>Group III</u>--restrained occupants, nurseries (nonambulatory).

<u>Group IV</u>--aircraft hangers, woodworking, factories, repair garages, service stations, storage garages, wholesale, general warehouse, printing plants, factories, ice plants, dwellings, hazardous flammable storage, less hazardous flammable storage.

<u>Group V</u>--private garages, sheds, barns.

The occupancy grouping in Table C8-4 represents that set used in the 1985 Edition of the *Provisions*. It resulted from a logical consolidation of Table C8-3, consideration of code enforcement problems, and the need to use a common hazard exposure grouping for all of the design provisions. The grouping and definition were modified in the 1988

Edition. It is felt that this grouping can be augmented as local conditions warrant. Specific consideration was given to Group III, essential facilities, to ensure that only those facilities specifically designated by the cognizant jurisdiction would be included because this determination has both political and economic impact.

Group II contains those occupancies that have large numbers of occupants either due to the overall size of the building or the number of stories; the character of the use, such as public assembly, schools, or colleges; or a height that exposes the occupants to greater life safety hazard. Other considerations included uses wherein the occupants were restrained or otherwise handicapped from moving freely, such as day care centers, hospitals, and jails.

Group I contains all uses other than those excepted generally from the provisions in Sec. 1.2. Those in Group I have lesser life hazard only insofar as there is the probability of lesser numbers of occupants in the buildings and the buildings are lower and/or smaller. The height of four stories was used in part due to the general model code use of this height as being the maximum allowable height for wood frame and masonry/wood frame classes of buildings.

In buildings with multiple uses, the building is to be assigned the classification of the highest group that occupies 15 percent or more of the total building area. Such assignments also should be considered when changes are made in the use of a building even though existing buildings are not within the scope of the *Provisions*. For example, if a portion subject to change of use is in a building of Seismic Hazard Exposure Group I, the portion represents 15 percent or more of the total building area and the use is found in Seismic Hazard Group II, then the entire building should be reclassified to Group II and the appropriate Seismic Performance Category applied based on the appropriate value of A_v and the Seismic Hazard Exposure Group II classification.

Consideration originally was given to reducing the number of groupings by combining Groups I and II and leaving Group III the same as is stated above. It was the consensus of those involved that such a merging would not be responsive to the relative life hazard problems.

1.4.3 Seismic Performance Categories

This section establishes the five design categories that are the keys for establishing requirements for any building based on its use (Seismic Hazard Exposure Group) and on the level of expected seismic ground motion (specifically, the coefficient A_v). Once the Seismic Performance Category (A, B, C, D, or E) for the building is established, many other requirements such as detailing, quality assurance, limitations, specialized requirements, and change of use are related to it.

The 1985 Edition of the *Provisions* contained four categories (A, B, C, and D) with Category B split for some materials. The 1988 Edition extended this to all materials and redesignated the categories as A, B, C, D, and E.

1.4.4 Category E Site Limitation

Essential facilities that may be required after an earthquake and that are located in zones of higher seismicity should not be located over an active fault. Although some structures could and may be designed to remain intact even if a fault occurs at the base, knowingly exposing an essential facility to such a risk is unreasonable and should be unnecessary.

1.5 ALTERNATE MATERIALS AND METHODS OF CONSTRUCTION

It is not possible for a design standard to provide criteria for the use of all possible materials and their combinations and methods of construction either existing or anticipated. While not citing specific materials or methods of construction currently available that require approval, this section serves to emphasize the fact that the evaluation and approval of alternate materials and methods require a recognized and accepted approval system. The requirements for materials and methods of construction contained within the document represent the judgment of the best use of the materials and methods based on well-established expertise. It is important that any replacement or substitute be evaluated with an understanding of all the ramifications of performance, strength, and durability implied by the *Provisions*.

It also is recognized that until needed approval standards and agencies are created, regulatory agencies will have to operate on the basis of the best evidence available to substantiate any application for alternates. It is strongly recommended that where there is an absence of accepted standards, applications be supported by extensive reliable data obtained from tests simulating, as closely as is practically feasible, the actual load and/or deformation conditions to which the material is expected to be subjected during the service life of the building. These conditions, where applicable, should include several cycles of full reversals of loads and deformations in the inelastic range.

1.6 QUALITY ASSURANCE

Earthquake-related building failures that are directly traceable to poor quality control during construction are innumerable. The literature is replete with reports pointing out that collapse may have been prevented had proper inspection been exercised.

The remarkable performance during earthquakes by California schools constructed since 1933 is due in part to the rigorous supervision of construction required by state law. Independent special inspection, approved and supervised by the Office of the State Architect, is an important feature of the California requirements. Such an excellent record of performance has influenced the writing of the *Provisions* so as to rely heavily on the concept of special inspection to ensure good construction.

Recognizing that there must be coordinated responsibility during construction, the *Provisions* set forth the role each party is expected to play in construction quality control. The building designer specifies the quality assurance requirements, the contractor exercises the control to achieve the desired quality, and the owner monitors the construction process through special inspection to protect the public interest in safety of buildings. Thus, the Special Inspector is the owner's inspector. It is essential that each party recognize his or her responsibilities, understand the procedures, and be capable of carrying them out. Because the contractor and the specialty subcontractors are doing the work and exercising control on quality, it is essential that the special inspection be performed by someone not in their direct employ and also be approved by the Regulatory Agency. When the owner is also the builder, he or she should engage independent agencies to conduct these inspections rather than try to qualify his his or her own employees.

The approach used in preparing the 1978 ATC provisions was to borrow liberally from the pattern already established by the 1976 *UBC*, which detailed structural quality provisions in Chapter 3, Sec. 305, Special Inspections. These were retained with minimal change in Chapter 3, Sec. 306, Special Inspections, of the 1985 *UBC*. There have been some changes in the 1988 Edition of the *Provisions* as well as in the 1988 *UBC*.

There are two major differences, however, between the ***NEHRP Recommended Provisions*** and the *UBC*. First the *Provisions* cover only those portions and components of the building that are directly affected by earthquake motions and whose response could affect life safety and continued functioning of the building (where designated). Second, the *Provisions* for the first time attempt to place minimum quality assurance requirements on installation of nonstructural components that are designated as deserving special attention during construction. These are described as "Designated Seismic Systems" throughout and are defined as being "the Seismic Resisting Systems and those architectural, electrical, and mechanical systems and their components that require special performance characteristics." This means that the designer most familiar with the requirements of each system must spell out in a Quality Assurance Plan those components that will require special inspection and tests during construction to assure their ability to perform satisfactorily during earthquakes.

These provisions are concerned with those components that affect the building performance during an earthquake and/or that may be adversely affected by earthquake motions as specified under other sections of the *Provisions*. The requirements under Sec. 1.6 are minimum and it could very well be the decision of the designers to include all phases of construction throughout the project under a Quality Assurance Plan. For many buildings, the additional cost to do so would be minimal. The primary method of achieving quality assurance is through the use of specially qualified inspectors approved by the Regulatory Agency and working on behalf of the owner. The number of such inspectors actually employed will vary widely depending on the size, complexity, and function of the building. These provisions permit the designer or his employee to perform these inspections as long as they are approved by the Regulatory Agency having jurisdiction and can demonstrate reasonable competence in the particular category of work they inspect.

1.6.1 Quality Assurance Plan

Introduced here is the concept that the Quality Assurance Plan must be prepared by the person responsible for the design of each seismic system subject to quality assurance whether it be architectural, electrical, mechanical, or structural in nature. The plan may be a very simple listing of those elements of each system that have been designated as being important enough to receive special inspection and/or testing. The extent and duration of inspection must be set forth as well as the specific tests and the frequency of testing.

Although some design professionals have expressed reluctance to assume this duty because of an assumed increase in potential liability, it has been demonstrated by the performance of schools in California earthquakes that the improved quality also acts to protect the professional. Furthermore, the design professional is the most qualified person to prepare such a plan since he is the most familiar with the design concept.

The Regulatory Agency, however, must approve the plan and must obtain from each responsible contractor a written statement that the contractor understands the requirements of the plan and will exercise control to obtain conformance. The exact methods of control are left up to the individual contractor subject to approval by the Regulatory Agency. However, special inspection of the work is required in specific situations to give the agency reasonable assurance that the approved drawings and specifications are followed.

1.6.2 Special Inspection

The requirements listed in this section from foundations through structural wood are basically the same as those currently requiring special inspection under the 1988 *UBC* and it is a premise of these provisions that there will be available an adequate supply of knowledgeable and experienced inspectors to draw upon for the structural categories of work. Special training programs may have to be developed and implemented for the nonstructural categories.

A Special Inspector (identified as the owner's inspector) is defined as a "person approved by the Regulatory Agency as being qualified to perform special inspection." As a guide to such agencies, it is contemplated that the Special Inspector may be one of the following:

1. A person employed and supervised by the design architect or engineer of record who is responsible for the design of the designated seismic system for which the Special Inspector is engaged.

2. A person employed by an approved inspection and testing agency who is under the direct supervision of a registered engineer also employed by the same agency.

3. A manufacturer or fabricator of components, equipment, or machinery who has been approved for manufacturing components meeting seismic safety standards and who maintains a quality control plan approved by the Regulatory Agency. Evidence of such approval must be clearly marked on each designated seismic system component shipped to the job site.

1.6.2.8 Architectural Components

It is anticipated that the minimum requirements for architectural components will be complied with when the Special Inspector is satisfied that the method of anchorage or fastening and the number, spacing, and types of fasteners actually used conform with the plans and specifications for the component installed. It is noted that such special inspection requirements are only for those components required to have superior (S) or good (G) performance (see Chapter 8).

1.6.2.9 Mechanical and Electrical Components

In addition to verification of the fastening and anchorage for mechanical and electrical components, it is anticipated that the Special Inspector will verify that the designated components are labeled to meet S or G performance standards as required in Chapter 8 and as established by the Regulatory Agency.

Close cooperation between the designer, manufacturers, Special Inspector, and Regulatory Agency must be exercised until all learn their respective roles and a definite inspection routine is established.

1.6.3 Special Testing

The specified testing of the structural materials follows procedures and tests long established by industry standards. A possible exception is masonry where there was no single nationally accepted standard encompassing all of the diversity of materials now being used in masonry construction until the appearance of ACI-ASCE 530-88. The acceptance criteria should be agreed upon prior to contract award.

1.6.4 Reporting and Compliance Procedures

The success of a quality assurance plan depends upon the intelligence and knowledge of the inspector and the accuracy and thoroughness of his reports. It should be emphasized that both the Special Inspector and the contractor are required to submit to the Regulatory Agency a final certification as to the adequacy of the completed work. The contractor, with his day-to-day knowledge of the installation, is in the best position to state whether or not all the construction has been completed in accordance with approved plans and specifications. To be fully aware, however, the contractor must institute a system of reporting within his or her organization that enables him or her to effectively practice quality control. The inspector can only attest to the work he or she has personally inspected and, therefore, acts more as an auditor or monitor of the quality control program exercised by the contractor.

1.6.5 Approved Manufacturers' Certification

Provision is made for the special approval of manufactured designated components. This arises because most mechanical or electrical equipment is manufactured off-site and is delivered to a job in its own container. The Special Inspector, being at the job site, cannot judge the adequacy of anchorage or the seismic resistance of the equipment contained therein and, in most instances, cannot be present during the off-site manufacturing. It is expected, therefore, that a system of approvals and labeling must be established by the Regulatory Agency in much the same way as labeling of fire doors is presently being done.

APPENDIX TO CHAPTER 1

Alternate Maps and Alternate Method for Establishing Design Ground Motions

This appendix introduces new maps (Figures 1-5 through 1-8) defining the seismic ground-shaking hazard and incorporates a few necessary changes in the expression of certain provisions so that the new maps might be appropriately used with the *NEHRP Recommended Provisions*. This presentation in an appendix is intended to encourage evaluation of the new maps and new procedures through use. Comment is encouraged.

As explained in the commentary for Sec. 1.4.1, the two maps in the 1985 Edition of the *Provisions* were developed during the ATC-3 project from a single map prepared by Algermissen and Perkins of the U.S. Geological Survey (USGS) in 1976. In 1982, Algermissen and co-workers published a set of probabilistic maps for both acceleration and velocity using three different exposure times (thus three different levels of probability of exceedance of the ground motions). The basic procedure for generation of these new maps was not greatly different from that used for the 1976 map. The major difference is that the map for ground velocity was computed from basic data rather than being extrapolated from the acceleration map. In preparation for the 1988 Edition of the *Provisions*, the Building Seismic Safety Council committee dealing with the definition of the ground-shaking hazard decided that the maps represented a step forward and recommended that a modified version be incorporated into the 1988 Edition. The maps and the modifications are described below.

The ground motion maps of the contiguous United States presented here are the expected maximum horizontal acceleration and velocity in rock for periods of interest (exposure times) of 50 and 250 years (average return period for the expected ground motions of 474 and 2372 years). The mapped accelerations and velocities have a 90 percent probability of not being exceeded in the appropriate exposure times. Rock is taken here to mean material having a shear wave velocity of between 0.75 and 0.90 km/sec (Algermissen and Perkins, 1976). For a more complete discussion of the seismotectonic model, seismicity data and probabilistic model used, see Algermissen et al. (1982).

This model (Algermissen et al., 1982) has been used to recompute, for the maps presented here, ground motion values that include uncertainty in ground motion attenuation and fault rupture length. The estimates of uncertainty for fault rupture length relationship used for the maps is that of Mark (1977). The acceleration attenuation for the western United States is that of Schnabel and Seed (1977) modified for the eastern United States by Algermissen and co-workers (1982). The velocity attenuation used in the preparation of the maps was developed by Perkins and Harding (1988) using a data set and methods of analysis similar to that of Schnabel and Seed (1977). McGuire and Shedlock

(1981) give a standard deviation for Mark's (1977) fault rupture relationship of log (rupture length L) = 0.52 and Schnabel and Seed's (1977) attenuation relationship of $\ln_e$ (attenuation) = 0.62.

As already noted, the data and probabilistic model used (Algermissen et al., 1982) is the same as for the present maps. There is a single exception, however. Seismic Source Zone 104 of the Algermissen et al. report (1982) has been used in a different manner in the computation of the maps presented here. The line source fault model used by Algermissen and co-workers (1982) to concentrate seismicity on the Ramapo fault within Zone 104 has not been used for the present maps. It now is believed that this source of seismicity is more difficult than previously modeled and, consequently, in the model used here, the seismicity within Zone 104 is distributed equally throughout the zone and not concentrated along the Ramapo fault (see p. 17, Figure 3, Algermissen et al., 1982).

The ground motion maps for Alaska also have been recomputed to include fault rupture length and attenuation variability using the data and probabilistic model of Thenhaus and co-workers (1982). The same standard deviations for fault rupture length and attenuation as used for the contiguous United States was used in the recomputation of the Alaska ground motion maps.

The ground motion maps for Hawaii and Puerto Rico included in Figures 1-5 and 1-6 are taken directly from Part 2 of the 1985 Edition of the *NEHRP Recommended Provisions*. The only modification of these maps is the conversion of the velocity contours from inches per second to centimeters per second to conform with units used on the other maps. The ground motion values shown for Hawaii and Puerto Rico do not represent the results of a particular probabilistic ground motion calculation but are weighted averages of the ground motion estimates available at the time of the ATC 3 study (1978). The mapped values, however, are in general agreement with recent studies of probabilistic ground motion in these areas. Also, new mapping was not done for the other island regions shown on the 1985 maps; the Aleutians, Guam, Tutuila, and the Virgin Islands.

The new maps represent a very significant change from the maps in the main body of the *Provisions* in the areas of highest seismic activity. Specifically, the values of acceleration and velocity on the new maps are much higher for regions close to major faults in California. Some of this results from refinement in the analytical and statistical models for ground motion, but much of the difference results from the fact that the maps included in the 1978 ATC-3 report and the 1985 Edition of the *Provisions* truncated the highest values of acceleration from that shown on the 1976 Algermissen-Perkins map. (Refer to the discussion under "Map of EPA" in Sec. 1.4.1 of the *Commentary* for an explanation of the rationale for truncating the higher values.)

Although values of acceleration greater than 40 percent gravity and values of velocity greater than 40 centimeters per second are not required by the "*Provisions*" for use in design, the new maps have acceleration and velocity contours as high as 80 percent of gravity and 80 centimeters per second, respectively, with indications of higher accelerations and velocities within those contours. The earlier maps have no contour greater than 40 percent of gravity and there is no indication of higher values within that contour. It should be noted that within the areas of the 80 percent gravity contour some of the values computed in developing the new maps are substantially higher than 80 percent, particularly for the map with a 250-year time exposure (Figure 1-7). The same is true for velocities on the velocity maps. The geographical areas within the highest contours are quite small and microzoning would be required to accurately portray the acceleration and velocity information.

After considerable discussion, the BSSC committees responsible for preparing the 1988 Edition of the *Provisions* decided to retain the concept of truncating the higher values of acceleration and velocity for use in structural analysis and design. The limit for peak acceleration is retained at 0.4g (40 percent of gravity). The limit for velocity is set in a fashion that results in approximately an 8 percent increase in design values for areas that would be truncated on both sets of maps. Comment on the concept and the specific values is encouraged.

The inclusion of the maps with the longer exposure periods is intended to allow users to develop some perspective on the issue of performance expected should a rare event occur. The ratio between ground motions for the two exposure periods is not constant: generally, in areas with low to moderate seismicity, the motions for the longer exposure period are a larger multiple of those for the shorter period than in areas with the highest seismicity.

Because the new maps expressed ground velocity in units of velocity rather than as velocity-related acceleration, some conversion is necessary to use the provisions. A_v is used for several purposes in the *Provisions*:

1. To provide a design coefficient,
2. To define the level of seismicity, and
3. To define the design spectrum.

The basic relation $A_v = 0.012v$ was developed in a study by Wu and Hanson (1987) for the first purpose.

Strict application of the basic relation for the second purpose would have resulted in division points for the old Seismicity Index at velocities of 4.2, 12.5, and 16.7 cm/sec, which correspond directly to A_v values of 0.05, 0.15, and 0.20. Direct use of these values would imply a precision not intended by the original use of A_v. Furthermore, an additional division point was necessary for the new set of Seismic Performance Categories used in the 1988 Edition. (Prior to 1988 there were four, not five, categories.) Given the opportunity to plot contours at reasonable intervals for interpolation and the changes in requirements for Seismic Performance Categories, it was decided to plot contours for velocities of 2.5, 5, 7.5, 10, 15, 20, 30, 40, 60, and 80 cm/sec and to make the values of 2.5, 5, 10, and 20 serve as the division lines for defining the level of seismicity in determining the Seismic Performance Categories (Table 1A-2). (Some of the intermediate contours are omitted where several lines are close and parallel.) The net effect of the new maps and new points for defining Seismic Performance Category is that for many sites application of the "Appendix to Chapter 1" will result in a different Seismic Performance Category than would be obtained from application of "Chapter 1" without the appendix. Such changes must be evaluated on a case-by-case basis.

With regard to the third purpose, the use of velocity in cm/sec in definition of the design spectrum (Eq. 4-2) in lieu of the velocity-related acceleration coefficient A_v is not a direct substitution. The study by Wu and Hanson pointed out that one of the premises of the original development of the equation--that the spectral response velocity was 2.5 times the effective peak ground velocity--was quite conservative. (The 2.5 is explained under "Ground Motion Parameters" in Sec. 1.4.1 of the *Commentary*.) Wu and Hanson developed the new equation

$$Cs = \frac{0.013vS}{RT^{2/3}}$$

based upon a ratio of response velocity to ground velocity of approximately 1.65 instead of 2.5. The 1.65 represents a mean value of the ratio for average soil profiles. Consideration of relations between response velocity, response acceleration, ground velocity, and ground acceleration was given for various types of soil profiles and for several sites with widely different levels of seismicity by Wu and Hanson in arriving at the new expression for C_s in the velocity region of the spectrum. No corresponding change was made in the acceleration region of the spectrum (periods less than about 0.5 sec). If a similar rationale (use of the mean value) were applied in the acceleration region, the difference would not be as dramatic.

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Chapter 2 Commentary

DEFINITIONS AND SYMBOLS

This chapter defines the terms and symbols used in the Provisions. Added for the 1988 Edition are definitions related to base shear, concentrically and eccentrically braced frames, the story drift ratio, story shear, torsion, and symbols A and v for use with the new maps and procedures presented in the "Appendix to Chapter 1."

Chapter 3 Commentary

STRUCTURAL DESIGN REQUIREMENTS

3.1 DESIGN BASIS

In these provisions, the design of a structure (sizing of individual members, connections, and supports) is based on the internal forces resulting from a linear elastic analysis using the prescribed forces. It assumes that the structure as a whole under these prescribed forces should not deform beyond a point of significant yield. The elastic deformations then are amplified to estimate the real deformations in response to the design ground motion. (The amplification is in Sec. 4.6.) Sec. 3.8 prescribes the story drift limits controlling the deformation in the inelastic range when the structure is subjected to the actual seismic forces that may be generated by the specified ground motion. This procedure differs from that in prior codes and design provisions wherein the prescribed loads, sizing, and drift limits were at service or working stress levels.

The term "significant yield" specifically is not the point where first yield occurs in any member but is defined as that level causing complete plastification of at least the most critical region of the structure (e.g., formation of the first plastic hinge in the structure). A structural steel frame of compact members is assumed to reach this point when a plastic hinge develops in the most critical member of the structure. A concrete frame reaches this significant yield in its response to the prescribed forces when at least one of the sections of its most critical component reaches its strength as set forth in Chapter 11. For other structural materials that do not have their sectional yielding capacities as easily defined, modifiers to working stress values are provided in the respective material sections (Chapters 9 and 12).

These provisions contemplate a seismic resisting system with redundant characteristics wherein overstrength above the level of significant yield is obtained by plastification at other points in the structure prior to the formation of a complete mechanism. For example, in the two-story bent in Figure C3-1, significant yield is the level where plastification occurs at the most critical joint shown as Joint 1 and as Point 1 on the load-deflection diagram. With increased loading, causing the formation of additional plastic hinges, the capacity increases (following the solid line) until a maximum is reached. The overstrength capacity obtained by this continued inelastic action provides the reserve strength necessary for the structure to resist the extreme motions of the actual seismic forces that may be generated by the specified ground motion. The dotted line in Figure C3-1 is the load-deflection curve including the P-delta effects. The dash-dot line is the elasto-plastic curve that results with certain systems and materials.

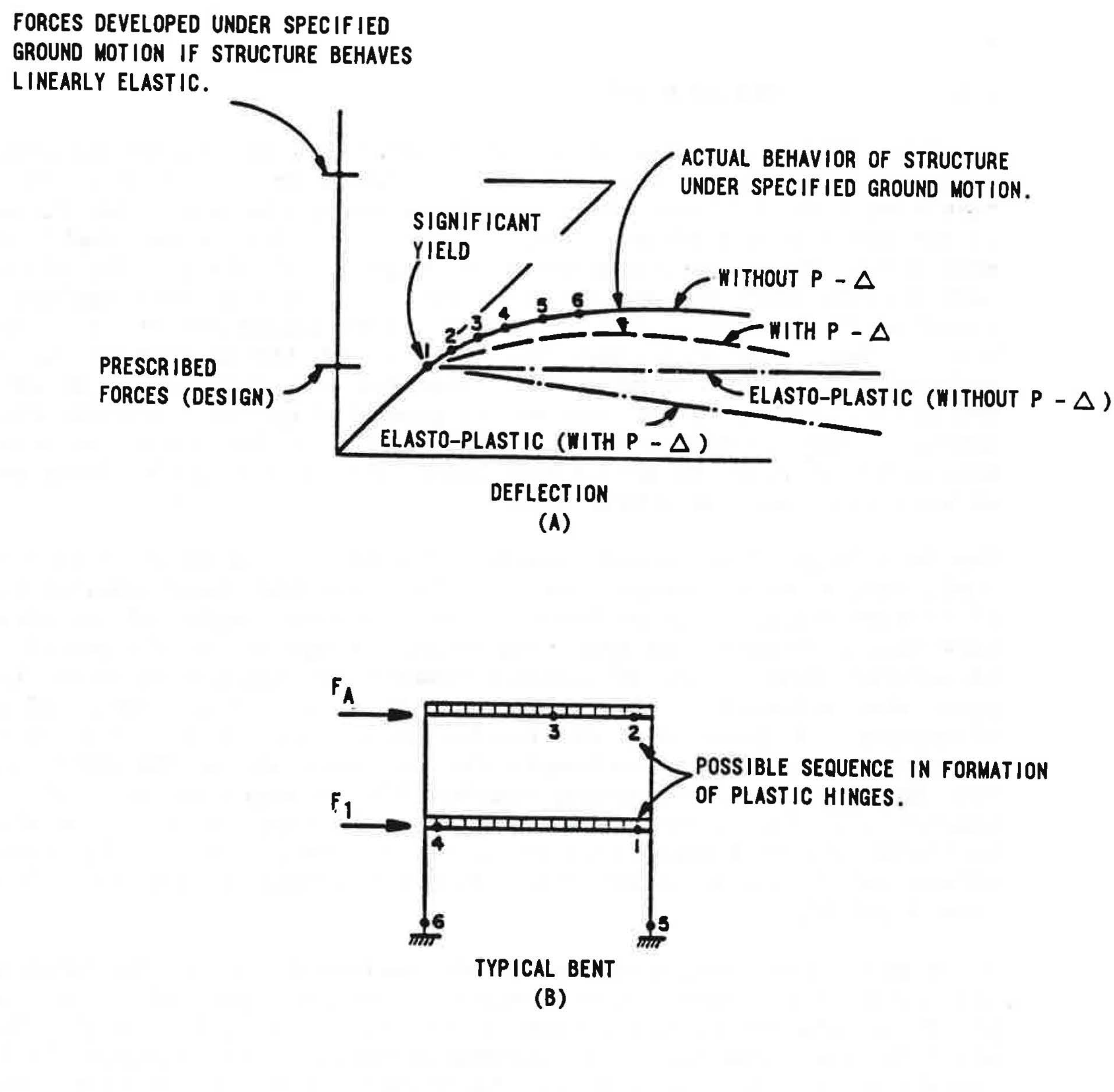


FIGURE C3-1
Formation of plastic hinges.

The response modification factor, R, and the Cd value for deflection amplification (Table 3-2), as well as the criteria for story drift including the P-delta effects, have been established considering that structures generally have additional overstrength capacity above that whereby the design loads cause significant yield. The R factor essentially represents the ratio of the forces that would develop under the specified ground motion if the structure behaved entirely linearly elastic to the prescribed design forces. The structure is to be designed so that the level of significant yield exceeds the prescribed design force. The ratio R is always larger then 1.0; thus, all structures are designed for forces smaller than the design ground motion would produce in a completely linear-elastic responding structure. This reduction is possible because of the actual energy absorption and energy dissipation capacity (toughness) that the whole structure possesses due to its capability to deform inelastically. This capacity is represented by the area under the actual load deformation curve. In establishing the R value, consideration also has been given to the performance of the different materials and systems in past earthquakes.

Note that the value of R increases with higher toughness and damping whereas the design seismic force decreases. R is used in the denominator of the term to calculate the design seismic force coefficient C_s (Eq. 4-2).

The values of R must be chosen and used with careful judgment. For example, lower values must be used for structures possessing a low degree of redundancy wherein all the plastic hinges required for the formation of a mechanism may be formed essentially simultaneously and at a force level close to the specified design strength. This situation can result in considerably more detrimental P-delta effects.

It should be noted that the design seismic coefficient C_s (Eq. 4-2) does not include a factor that varies for different types of occupancies. This point reflects the belief that increasing the forcing function alone does not necessarily increase the performance and is discussed more fully later in this commentary. The improved performance characteristics desired for more critical occupancies are provided by the design and detailing requirements set forth in Sec. 3.6 for each Seismic Performance Category and the more stringent drift limits in Table 3-5.

Sec. 3.1 in effect calls for the seismic design to be complete and in accordance with the principles of structural mechanics. The loads must be transferred rationally from their point of origin to the final points of resistance. This should be obvious but is often overlooked by those inexperienced in earthquake engineering.

3.2 SITE EFFECTS

The "Chapter 1 Commentary" for Sec. 1.4.1 presents the discussion applicable to Sec. 3.2.1 and 3.2.2. The "Appendix to Chapter 6" and its commentary provide background for Sec. 3.2.3, Soil-Structure Interaction.

Sec. 1.4.4 presents site limitations for buildings assigned to Seismic Performance Category E. Critical structures needed after a disaster and located in zones of higher seismicity should not be located over an active fault. Although it is known that some structures could and must be designed to remain intact even if a fault surface rupture goes through their bases, it is inappropriate for critical facilities to be so located.

3.3 FRAMING SYSTEMS

For purposes of these seismic analyses and design provisions, framing systems for buildings are grouped into four general categories of structural systems as shown in Table 3-2. These categories are similar to those contained for many years in the provisions of the *Uniform Building Code*; however, a further breakdown is included for various types of vertical components of the seismic resisting system.

In selecting the structural system, the designer is cautioned to consider carefully the interrelationship between continuity, toughness (including minimizing brittle behavior), and redundancy in the structural framing system as is subsequently discussed in this commentary.

Selection of R factors requires considerable judgment based on knowledge of actual earthquake performance as well as research studies; yet, they have a major effect on building costs. The factors in Table 3-2 continue to be reviewed in light of recent research results in order to ensure the most appropriate values are used.

In the selection of the R values for the various systems, consideration was given to the general observed performance of each of the system types during past earthquakes, the general toughness (ability to absorb energy without serious degradation) of the system, and the general amount of damping present in the system when undergoing inelastic response. The designer is cautioned to be especially careful in detailing the more brittle types of systems (low C_d values).

3.3.1 Classification of Framing Systems

A Bearing Wall System refers to that structural support system wherein major load-carrying columns are omitted and the wall and/or partitions are of sufficient strength to carry the gravity loads for some portion of the building (including live loads, floors, roofs, and the weight of the walls themselves). The walls and partitions supply, in plane, lateral stiffness and stability to resist wind and earthquake loadings as well as any other lateral loadings. In some cases, vertical trusses are employed to augment lateral stiffness.

In general, this system has comparably lower values of R than the other systems due to the frequent lack of redundancy for the vertical and horizontal load support. The category designated "light framed walls with shear panels" was intended to cover wood or steel stud wall systems with finishes other than masonry veneers.

A Building Frame System is similar to the "vertical load-carrying frame" system described in the 1976 Structural Engineers Association of California (SEAOC) recommendations. In order to qualify for this system, the gravity loads should be carried primarily by a frame supported on columns rather than by bearing walls. Some minor portions of the gravity load can be carried on bearing walls but the amount so carried should not represent more than a few percent of the building area. Lateral resistance is provided by nonbearing structural walls or braced frames. The light framed walls with shear panels are intended only for use with wood and steel building frames. Although there is no requirement to provide lateral resistance in this framing system, it is strongly recommended that some moment resistance be incorporated at the joints. In a structural steel frame, this could be in the form of top and bottom clip angles or tees at the beam- or girder-to-column connections. In reinforced concrete, continuity and full anchorage of longitudinal steel and stirrups over the length of beams and girders framing into columns would be a good design practice. With this type of interconnection, the frame provides a nominal secondary line of resistance even though the components of the seismic resisting system are designed to carry all the seismic force.

A Moment Resisting Space Frame System is a system having an essentially complete space frame as in the Building Frame System. However, in this system, the lateral resistance is provided by moment resisting frames composed of columns with interacting beams or girders. The moment resisting frames may be either Ordinary, Intermediate, or Special Moment Frames as indicated in Table 3-2 and limited by the Seismic Performance Categories.

Special Moment Frames must meet all of the design and detail requirements of Sec. 10.7 or Sec. 11.5 and the sections referred to therein. The ductility requirements for these frame systems are required in areas where high seismic hazards are anticipated; see Table 1-1. Intermediate Moment Frames of concrete must meet the requirements of Sec. 11.4. For buildings in which these special design and detailing requirements are not used, lower R values are specified, indicating that ordinary framing systems do not possess as much toughness and that less reduction from the elastic response can be tolerated. Note that

Sec. 3.3.4 requires that any moment frames in Categories D or E be "Special Moment Frames."

A Dual System consists of a three-dimensional space frame made up of columns and beams that provides primary support for the gravity loads. Lateral resistance is supplied by structural nonbearing walls or bracing; the frame is provided with a redundant lateral force system that is a Moment Frame complying with the requirements of Sec. 10.7 and Sec. 11.4 or 11.5. The Moment Frame is required to be capable of resisting at least 25 percent (judgmentally selected) of the specified seismic force. Normally the Moment Frame would be a part of the basic space frame. The walls or bracing acting together with the Moment Frame must be capable of resisting all of the design seismic force.

The following analyses are required for Dual Systems:

1. The frame and shear walls or braced frames must resist the prescribed lateral seismic force in accordance with the relative rigidities considering fully the interaction of the walls or braced frames and the moment frames as a single system. This analysis must be made in accordance with the principles of structural mechanics considering the relative rigidities of the elements and torsion in the system. Deformations imposed upon members of the *moment* frame by the interaction with the shear walls or braced frames must be considered in this analysis.

2. The Moment Frame must be designed to have a capacity to resist at least 25 percent of the total required lateral seismic force including torsional effects.

Inverted Pendulum Structures are singled out for special consideration because of their unique characteristics and because they are often associated with buildings. Frequently overlooked design aspects and field experience make it desirable to give these structures special attention.

3.3.2 Combinations of Framing Systems

For those cases where combinations of structural systems are employed, the designer must use judgment in selecting the appropriate R and Cd values. The intent of Sec. 3.3.2.1 is to prohibit support of one system by another possessing characteristics that result in a lower base shear factor. The entire system should be designed for the higher seismic shear as the provision stipulates. The exception is included to permit the use of such systems as a braced frame penthouse on a moment frame building in which the mass of the penthouse does not represent a significant portion of the total building and, thus, would not materially affect the overall response to earthquake motions.

Sec. 3.3.2.2 pertains to details and is included to help ensure that the more ductile details inherent with the design for the higher R value system will be employed throughout. The intent is that details common to both systems be designed to remain functional throughout the response in order to preserve the integrity of the seismic resisting system.

3.3.3-3.3.5 SEISMIC PERFORMANCE CATEGORIES A, B, C, D, AND E

General framing system requirements for the five building Seismic Performance Categories A, B, C, D, and E are given in these sections. The corresponding design and detailing requirements are given in Sec. 3.6 and Chapters 9 through 12. Any type of building framing system permitted by the provisions may be used for Categories A, B, and C except frames limited to Category A or Categories A and B only by the requirement of Chapters 11 and 12. Limitations regarding the use of different structural systems are given for Categories D and E.

Sec. 3.3.4 covers Category D, which compares roughly to the present California design practice for normal buildings other than hospitals. According to the requirements of Chapters 10 and 11, all moment-resisting frames of steel or concrete must be Special Moment Frames. Note that present SEAOC and *UBC* recommendations have similar requirements for concrete frames; however, Ordinary Moment Frames of structural steel may be used for heights up to 160 feet (48.6 m). In keeping with the philosophy of present codes for zones of high seismic risk, these provisions continue limitations on the use of certain types of structures over 160 feet (48.6 m) in height but with some changes. Although it is agreed that the lack of reliable data on the behavior of high--rise buildings whose structural systems involve shear walls and/or braced frames makes it convenient at present to establish some limits, the values of 160 feet (48.6 m) and 240 feet (73.1 m) introduced in these provisions are arbitrary. Considerable disagreement exists regarding the adequacy of these values, and it is intended that these limitations be the subject of further study.

These provisions require that buildings in Category D over 160 feet (48.6 m) in height have one of the following seismic resisting systems:

1. A moment resisting frame system with Special Moment Frames capable of resisting the total prescribed seismic force. This requirement is the same as present SEAOC and *UBC* recommendations.

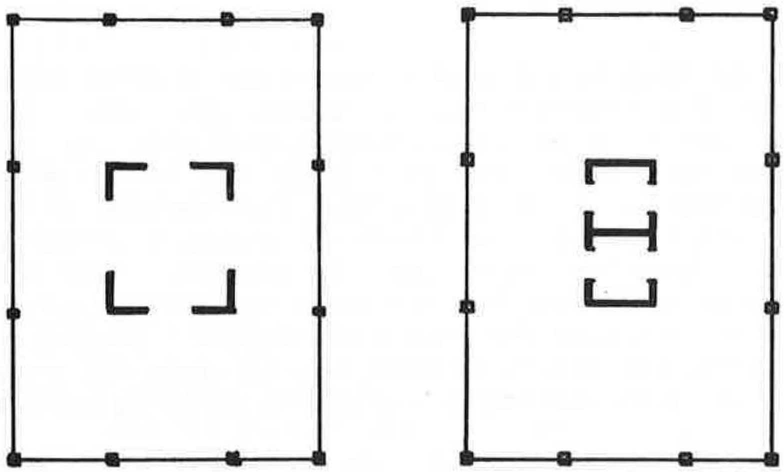
2. A Dual System as defined in Sec. 2.1, wherein the prescribed forces are resisted by the entire system and the Special Moment Frame is designed to resist at least 25 percent of the prescribed seismic force. This requirement is also similar to present SEAOC and *UBC* recommendations. The purpose of the 25

percent frame is to provide a secondary defense system with higher degrees of redundancy and ductility in order to improve the ability of the building to support the service loads (or at least the effect of gravity loads) after strong earthquake shaking. It should be noted that SEAOC and *UBC* provisions prior to 1987 required that shear walls or braced frames be able to resist the total required seismic lateral forces independently of the Special Moment Frame. These provisions require only that the true interaction behavior of the frame-shear wall (or braced frame) system be considered (see Table 3-2). If the analysis of the interacting behavior is based only on the seismic lateral force vertical distribution recommended in the Equivalent Lateral Force Procedure of Chapter 4, the interpretation of the results of this analysis for designing the shear walls or braced frame should recognize the effects of higher modes of vibration. The internal forces that can be developed in the shear walls in the upper stories can be more severe than those obtained from such analysis.

3. The use of a shear wall (or braced frame) system of cast-in-place concrete or structural steel up to a height of 240 feet (73.1 m) is permitted if, and only if, braced frames or shear walls in any plane do not resist more than 33 percent of the seismic design force including torsional effects. The intent is that each of these shear walls or braced frames be in a different plane and that the four or more planes required be spaced adequately throughout the plan or on the perimeter of the building in such a way that the premature failure of one of the single walls or frames will not lead to excessive inelastic torsion.

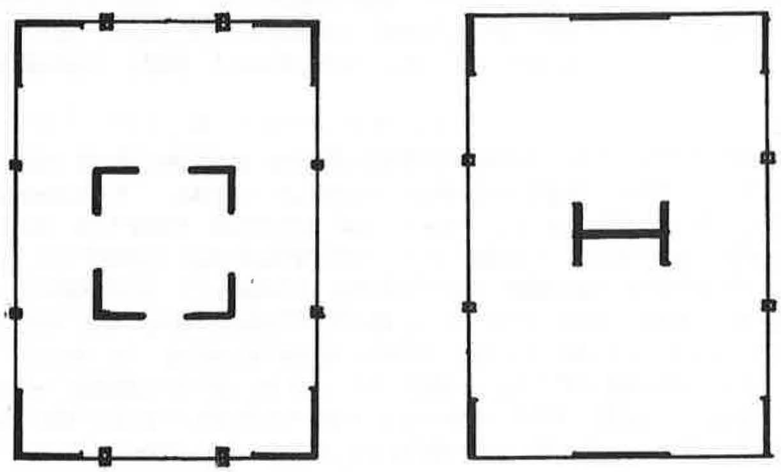
Although the structural system with lateral resistance concentrated in the interior core, as indicated in Figure C3-2, is acceptable according to the provisions, it is highly recommended that use of such a system be avoided, particularly for taller buildings. The intent is to replace it by the system with lateral resistance distributed across the entire building, as shown in Figure C3-3. The latter system is believed to be more suitable in view of the lack of reliable data regarding the behavior of tall buildings having structural systems based on central cores formed by coupling shear walls or slender braced frames.

Sec. 3.3.5 covers Category E, which is restricted to essential facilities in zones of relatively high seismicity. Because of the necessity for reducing risk (particularly in terms of protecting life safety or maintaining function by minimizing damage to nonstructural building elements, contents, equipment, and utilities), the height limitations for Category E are reduced. Again, the limits--100 feet (30.5 m) and 160 feet (48.6 m)--are arbitrary and require further study. The developers of these provisions believe that, at present, it is advisable to establish these limits, but the importance of having more



(A) (B)

FIGURE C3-2
Arrangement of shear walls and braced frames--not recommended.
Note that the heavy lines indicate shear walls and/or braced frames.



(A) (B)

FIGURE C3-3
Arrangement of shear walls and braced frames--recommended.
Note that heavy lines indicate shear walls and/or braced frames.

stringent requirements for detailing the seismic resisting system as well as the nonstructural components of the building must be stressed. Such requirements are specified in Sec. 3.6 and 3.7 and Chapters 9 through 12.

The response of a building will depend not only on the structural elements that the designer has calculated but rather on all elements, structural and nonstructural, calculated or not. In the initial stages of a large earthquake, the base shear and the distribution of shear throughout the height of a building, for example, will be distributed to both structural and nonstructural elements strictly in accordance with their effective rigidities. In essence, rigid elements that are physically divorced from the structure by flexible connections will not be reliably effective for resisting shears. However, some stiffness due to friction or the force necessary to cause the connections to bend will contribute to the shortening of the building period.

The enclosing of the space frame by rigid nonstructural components materially changes the distribution of the internal forces of the structure. For example, if a fairly strong nonstructural partition is rigidly attached to a moment resisting frame, the frame bent will act as a shear wall until failure of the partition occurs. As a shear wall, it will resist more load than the designer assumed, with higher overturning stresses, different diaphragm shears, etc. In some earthquakes, this uncalculated redistribution of forces has caused structural components to fail before the nonstructural partitions failed. Equation 4-5 (for period) in Sec. 4.2.2 partially accounts for this stiffening effect since it is based on observations of actual buildings before, during, and after earthquakes. Any stiffening effect in the building due to nonstructural components must be accounted for in the period determination of the structure and, consequently, in the design.

In many buildings, the seismic resisting system does not include all of the components that support the gravity loads. A common example would be a flat-slab concrete warehouse of several stories in height in which the lateral seismic loads are resisted by exterior shear walls or exterior ductile moment resisting frames. Ordinarily the internal slabs and columns that resist gravity loads are not designed to resist lateral seismic loads since their resistance is small in comparison with the resistance of the exterior walls or frames. However, although they are not needed for lateral resistance, they do deform with the rest of the structure as it deforms under lateral loads.

Sec. 3.3.4.3 requires that the vertical load-carrying capacity be reviewed at the actual deformations resulting from the earthquake. In the example of the flat-slab warehouse, there will be bending moments in the columns and slabs and an uneven shear distribution at the column capitals. At the calculated deflections (using C_d as noted elsewhere) and the resulting imposed moments and shears, it must be demonstrated that the members and connections will not fail under the design gravity loadings. The loading is cyclical so static ultimate load capacities may not be reached. If the combination of these loads and deformations results in stresses below yield, it can be assumed that the system is capable of supporting the gravity loads. If the stresses are above yield, sufficient ductility under cyclic loading must be provided. If the gravity load-bearing system is to provide any calculated resistance to the seismic resisting system (no matter how small), the detailing for ductility must be consistent with the values given in Table 3-2. In the example of the flat-slab warehouse, the connections can still carry the design gravity loadings if they satisfy the requirements of Sec. 11.5.

3.4 BUILDING CONFIGURATION

The configuration of a building can significantly affect its performance during a strong earthquake that produces the ground motion contemplated in the *Provisions*. Configuration can be divided into two aspects, plan configuration and vertical configuration. The *Provisions* were basically derived for buildings having regular configurations. Past earthquakes have repeatedly shown that buildings having irregular configurations suffer greater damage than buildings having regular configurations. This situation prevails even with good design and construction. These provisions are designed to encourage that buildings be designed to have regular configurations.

The addition of Tables 3-3 and 3-4 in the 1988 Edition provides, in the *Provisions* volume itself, definitions of irregularities that previously were covered only in the *Commentary*. The definitions are clearer and, therefore, should be easier to enforce than the vague criteria presented in the 1985 Edition.

Sec. 3.4.1 indicates, by reference to Table 3-3, when a building must be designated as having a plan irregularity for the purposes of the *Provisions*.

A building may have a symmetrical geometric shape without re-entrant corners or wings but still be classified as irregular in plan because of distribution of mass or vertical seismic resisting elements. Torsional effects in earthquakes can occur even when the static centers of mass and resistance coincide. For example, ground motion waves acting with a skew with respect to the building axis can cause torsion. Creating or yielding in a nonsymmetrical fashion also can cause torsion. These effects also can magnify the torsion due to eccentricity between the static centers. For this reason, buildings having an eccentricity between the static center of mass and the static center of resistance in excess of 10 percent of the building dimension perpendicular to the direction of the seismic force should be classified as irregular. The vertical resisting components may be arranged so that the static centers of mass and resistance are within the limitations given above and still be unsymmetrically arranged so that the prescribed torsional forces would be unequally distributed to the various components.

There is a second type of distribution of vertical resisting components that, while not being classified as irregular, does not perform well in strong earthquakes. This arrangement is termed a core-type building with the vertical components of the seismic resisting system concentrated near the center of the building. Better performance has been observed when the vertical components are distributed near the perimeter of the building. In recognition of the problems leading to torsional instability, a torsional amplification factor is introduced in Section 4.4.1.

A building having a regular configuration can be square, rectangular, or circular. A square or rectangular building with minor re-entrant corners would still be considered regular but large re-entrant corners creating a cruciform form would be classified as an irregular configuration. The response of the wings of this type of building is generally different than the response of the building as a whole, and this produces higher local forces than would be determined by application of the *Provisions* without modification. Other plan configurations such as H-shapes that have a geometrical symmetry also would be classified as irregular because of the response of the wings.

Significant differences in stiffness between portions of a diaphragm at a level are classified as irregularities since they may cause a change in the distribution of seismic forces to the vertical components and create torsional forces not accounted for in the normal distribution considered for a regular building. Examples of plan irregularities are illustrated in Figure C3-4.

Where there are discontinuities in the lateral force resistance path, the structure can no longer be considered to be "regular." The most critical of the discontinuities to be considered is the out-of-plane offset of vertical elements of the seismic force resisting elements. Such offsets impose vertical and lateral load effects on horizontal elements that are, at the least, difficult to provide for adequately.

Where vertical elements of the lateral force resisting system are not parallel to or symmetric with major orthogonal axes, the static lateral force procedures of the *Provisions* cannot be applied as given and, thus, the structure must be considered to be "irregular."

Sec. 3.4.2 indicates, by reference to Table 3-4, when a structure must be considered to have a vertical irregularity. Vertical configuration irregularities affect the responses at the various levels and induce loads at these levels that are significantly different from the distribution assumed in the Equivalent Lateral Force Procedure given in Chapter 4.

A moment resisting frame building might be classified as having a vertical irregularity if one story were much taller than the adjoining stories and the resulting decrease in stiffness that would normally occur was not, or could not be, compensated for. Examples of vertical irregularities are illustrated in Figure C3-5.

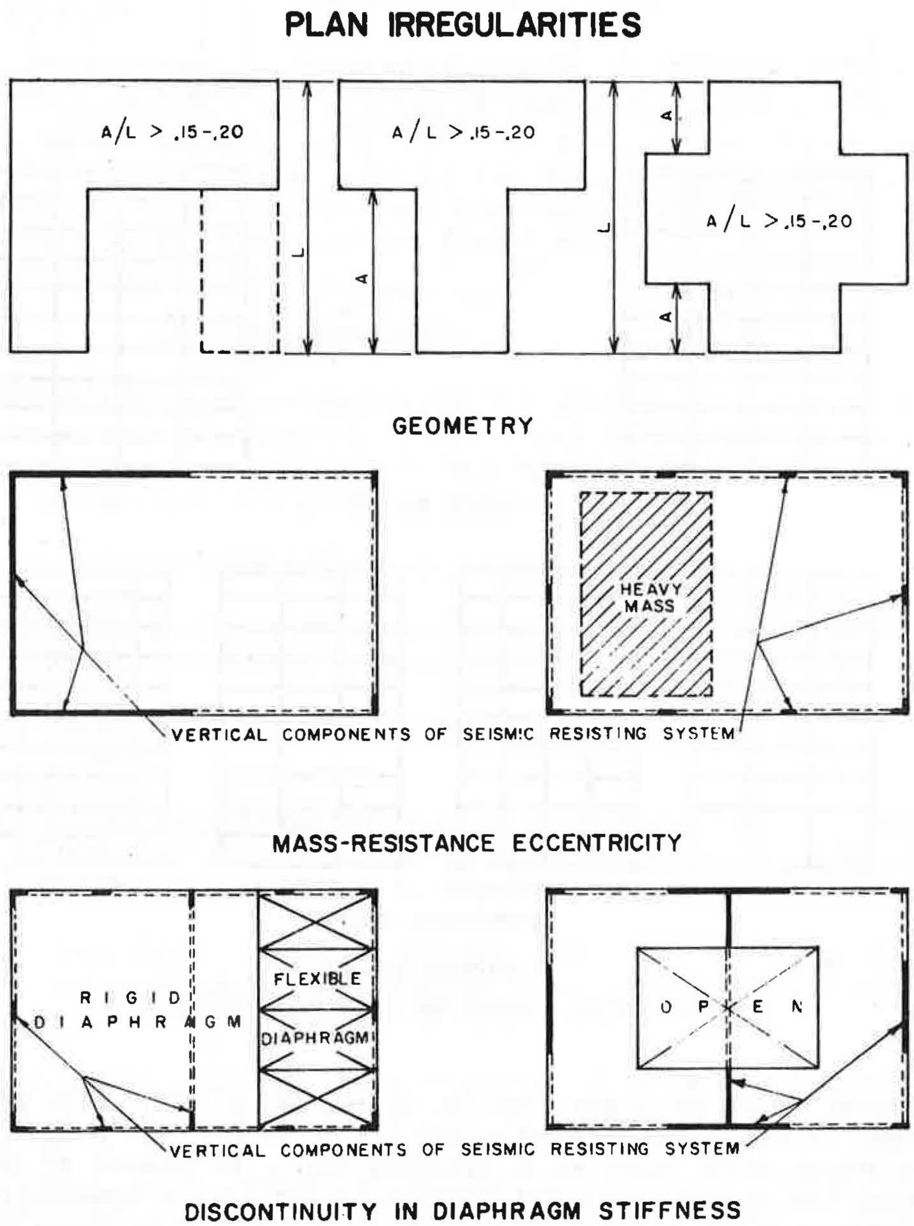


FIGURE C3-4
Building plan irregularities.

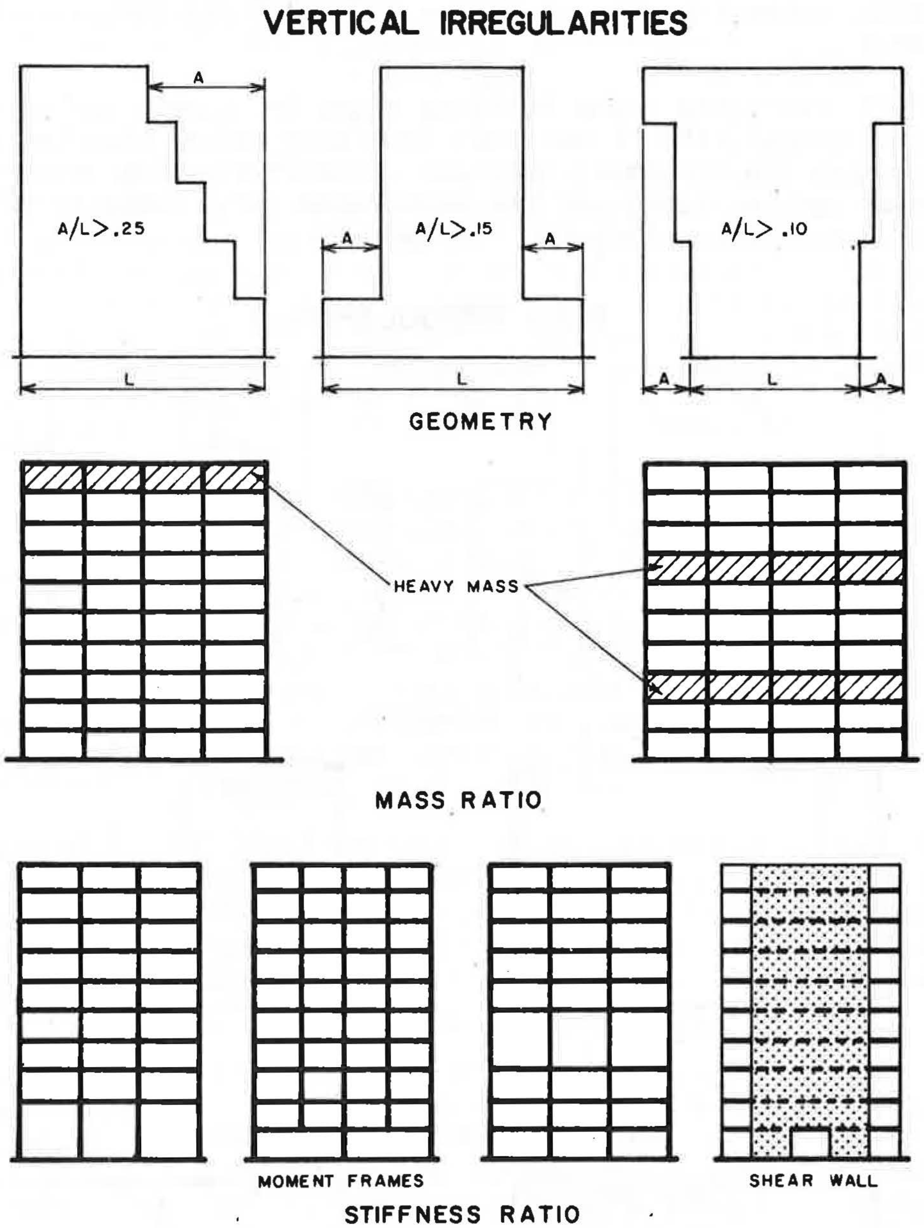


FIGURE C3-5
Building elevation irregularities.

A building would be classified as irregular if the ratio of mass to stiffness in adjoining stories differs significantly. This might occur when a heavy mass, such as a swimming pool, is placed at one level. Note that the exception in the *Provisions* provides a comparative stiffness ratio between stories to exempt structures from being designated as having a vertical irregularity of the types specified.

One type of vertical irregularity is created by unsymmetrical geometry with respect to the vertical axis of the building. The building may have a geometry that is symmetrical about the vertical axis and still be classified as irregular because of significant horizontal offsets in the vertical elements of the lateral force resisting system at one or more levels. An offset is considered to be significant if the ratio of the larger dimension to the smaller dimension is more than 130 percent. The building also would be considered irregular if the smaller dimension were below the larger dimension, thereby creating an inverted pyramid effect.

The designation of weak story irregularity has been added to those previously considered. The problem of concentration of energy demand in the resisting elements in a story as a result of abrupt changes in strength capacity between stories has been noted in past earthquakes. Note that an exception has been provided in Sec. 3.7.3 when there is considerable overstrength of the "weak" story.

3.5 ANALYSIS PROCEDURES

Many of the standard procedures for the analysis of forces and deformations in buildings subjected to earthquake ground motion, including the two procedures specified in the *Provisions*, are listed below in order of increasing rigor and expected accuracy:

1. Equivalent Lateral Force Procedure (Chapter 4).

2. Modal Analysis Procedure with one degree of freedom per floor in the direction being considered (Chapter 5).

3. Modal Analysis Procedure with several degrees of freedom per floor.

4. Inelastic Response History Analysis involving step-by-step integration of the coupled equations of motion with one degree of freedom per floor in the direction being considered.

5. Inelastic Response History Analysis involving step-by-step integration of the coupled equations of motion with several degrees of freedom per floor.

Each procedure becomes more rigorous if effects of soil-structure interaction are considered, either as presented in the "Appendix to Chapter 6" or through a more complete analysis of this interaction as appropriate. Every procedure improves in rigor if combined with use of results from experimental research (not described in these design provisions).

Sec. 3.5

The Equivalent Lateral Force (ELF) Procedure specified in Chapter 4 is similar in its basic concept to past SEAOC recommendations (1968, 1973, and 1974), but several improved features have been incorporated.

The modal superposition method (Newmark and Rosenblueth, 1971; Clough and Penzien, 1975; Thomson, 1965; Wiegel, 1970) is a general procedure for linear analysis of the dynamic response of structures. In various forms, modal analysis has been widely used in the earthquake-resistant design of special structures such as very tall buildings, offshore drilling platforms, dams, and nuclear power plants, but this is the first time that modal analysis has been included in design provisions for buildings. The Modal Analysis Procedure specified in Chapter 5 is simplified from the general case by restricting consideration to lateral motion in a plane. Only one degree of freedom is required per floor for this type of motion.

The ELF Procedure of Chapter 4 and the Modal Analysis Procedure of Chapter 5 are both based on the approximation that the effects of yielding can be adequately accounted for by linear analysis of the seismic resisting system for the design spectrum, which is the elastic acceleration response spectrum reduced by the response modification factor, R. The effects of the horizontal component of ground motion perpendicular to the direction under consideration in the analysis, the vertical component of ground motion, and torsional motions of the structure are all considered in the same simplified approaches in the two procedures. The main difference between the two procedures lies in the distribution of the seismic lateral forces over the height of the building. In the Modal Analysis Procedure, the distribution is based on properties of the natural vibration modes, which are determined from the actual mass and stiffness distribution over the height. In the ELF Procedure, the distribution is based on simplified formulas that are appropriate for regular buildings as specified in Sec. 3.4 and 3.5. Otherwise, the two procedures are subject to the same limitations.

Either of the two analytical procedures is likely to be inadequate if the lateral motions in two orthogonal directions and the torsional motion are strongly coupled. Such would be the case if the building were irregular in its plan configuration (see Sec. 3.4) or if it had a regular plan but its lower natural frequencies were nearly equal and the centers of mass and resistance were nearly coincident. A general model for the analysis of such buildings would include at least three degrees of freedom per floor--two translational and one torsional motion. Such a structure usually would have many modes that show a combination of translational and torsional motion. Analysis procedures similar to those specified in Chapter 5 can be applied to buildings of this type, with suitable generalization of the concepts involved. It is necessary, for example, to account for the facts that a given mode might be excited by both horizontal components of ground motion and modes that are primarily torsional can be excited by the translational components of the ground-shaking.

The methods of modal analysis can be generalized further to model the effect of diaphragm flexibility, soil-structure interaction, etc. In the most general form, the idealization would take the form of a large number of mass points, each with six degrees of freedom (three translation and three rotational) connected by generalized stiffness elements.

The ELF Procedure (Chapter 4) and both versions of the Modal Analysis Procedure (the simple version given in Chapter 5 and the general version with several degrees of freedom per floor mentioned above) are all likely to err systematically on the unsafe side if story strengths are distributed irregularly over height. This feature is likely to lead to concentration of ductility demand in a few stories of the building. A simple procedure to account for irregular strength distribution is discussed below in the commentary for Sec. 3.7.3.

The actual strength properties of the various components of a building can be explicitly considered only by a nonlinear analysis of dynamic response by direct integration of the coupled equations of motion. This method has been used extensively in research studies of earthquake response of yielding structures. If the two lateral motions and the torsional motion are expected to be essentially uncoupled, it would be sufficient to include only one degree of freedom per floor, the motion in the direction along which the building is being analyzed; otherwise at least three degrees of freedom per floor, two translational motions and one torsional, should be included. It should be recognized that the results of a nonlinear response history analysis of such mathematical building models are only as good as are the models chosen to represent the building vibrating at amplitudes of motion large enough to cause significant yielding during strong ground motions. Furthermore, reliable results can be achieved only by calculating the response to several ground motions--recorded accelerograms and/or simulated motions--and examining the statistics of response.

It is possible with presently available computer programs to perform two-dimensional inelastic analyses of reasonably symmetrical structures. The intent of such analyses could be to estimate the sequence in which components become inelastic and to indicate those components requiring strength adjustments so as to remain within the required ductility limits. It should be emphasized that with the present state of the art in elastic analysis, there is no one method that can be applied to all types of buildings. Further, the reliability of the analytical results are sensitive to:

- The number and appropriateness of the time-histories of input motion,
- The practical limitations of mathematical modeling including interacting effects of nonelastic elements,

- The nonlinear algorithms, and
- The assumed hysteretic behavior.

Because of these sensitivities and limitations, the maximum base shear produced in the inelastic analysis should not be less than that required by Chapter 5.

The least rigorous analytical procedure that may be used in determining the design earthquake forces and deformations in buildings depends on the Seismic Performance Category and the structural characteristics (in particular, regularity). Regularity is defined in Sec. 3.4.

Buildings in Seismic Performance Category A are not required to be analyzed as a whole for seismic forces. Certain minimum requirements are given elsewhere in the *Provisions*. For the higher Seismic Performance Categories, the ELF Procedure is the minimum level of analysis except that a more rigorous procedure is required for Category D or E buildings classified as irregular. The Modal Analysis Procedure adequately addresses vertical irregularities of stiffness, mass, or geometry. Other irregularities must be carefully considered.

The basis for the ELF Procedure and its limitations were discussed above. It is adequate for most regular buildings; however, the designer may wish to employ a more rigorous procedure (see list of procedures at beginning of this section for those regular buildings where it may be inadequate).

The ELF Procedure is likely to be inadequate in the following cases:

- Buildings with irregular mass and stiffness properties in which case the simple equations for vertical distribution of lateral forces (Eq. 4-6 and 4-6a) may lead to erroneous results;
- Buildings (regular or irregular) in which the lateral motions in two orthogonal directions and the torsional motion are strongly coupled; and
- Buildings with irregular distribution of story strengths leading to possible concentration of ductility demand in a few stories of the building.

In such cases, a more rigorous procedure that considers the dynamic behavior of the structure should be employed.

Buildings with certain types of vertical irregularities may be analyzed as regular buildings in accordance with the provisions of Chapter 4.

These buildings are generally referred to as setback buildings. The following procedure may be used:

1. The base and tower portions of a building having a setback vertical configuration may be analyzed as indicated in (2) below if:

   a. The base portion and the tower portion, considered as separate buildings, can be classified as regular <u>and</u>

   b. The stiffness of the top story of the base is at least five times that of the first story of the tower.

   When these conditions are not met, the building must be analyzed in accordance with Chapter 5.

2. The base and tower portions may be analyzed as separate buildings in accordance with the following:

   a. The tower may be analyzed in accordance with the procedures in Chapter 4 with the base taken at the top of the base portion.

   b. The base portion then must be analyzed in accordance with the procedures in Chapter 4 using the height of the base portion of h_n and with the gravity load and base shear seismic forces of the tower portion acting at the top level of the base portion.

The design provisions in Chapter 5 include a simplified version of modal analysis that accounts for irregularity in mass and stiffness distribution over the height of the building. It would be adequate, in general, to use the ELF Procedure for buildings whose floor masses and cross-sectional areas and moments of inertia of structural members do not differ by more than 30 percent in adjacent floors and in adjacent stories.

For other buildings, the following procedure should be used to determine whether the Modal Analysis Procedures of Chapter 5 should be used:

1. Compute the story shears using the ELF Procedure specified in Chapter 4.

2. On this basis, approximately dimension the structural members, and then compute the lateral displacements of the floor.

3. Replace h_x^k in Eq. 4-6a with these displacements, and recompute the lateral forces to obtain the new story shears.

4. If at any story the recomputed story shear differs from the corresponding value as obtained from the procedures of Chapter

4 by more than 30 percent, the building should be analyzed using the procedure of Chapter 5. If the difference is less than this value, the building may be designed using the story shear obtained in the application of the present criterion and the procedures of Chapter 5 are not required.

Application of this procedure to these buildings requires far less computational effort than the use of the Modal Analysis Procedure of Chapter 5 and, in the majority of the buildings, use of this procedure will determine that modal analysis need not be used and will also furnish a set of story shears that practically always lie much closer to the results of modal analysis than the results of the ELF Procedure.

This procedure is equivalent to a single cycle of Newmark's method for calculation of the fundamental mode of vibration. It will detect both unusual shapes of the fundamental mode and excessively high influence of higher modes. Numerical studies have demonstrated that this procedure for determining whether modal analysis must be used will, in general, detect cases that truly should be analyzed dynamically; however, it generally will not indicate the need for dynamic analysis when such an analysis would not greatly improve accuracy.

3.6 DESIGN AND DETAILING REQUIREMENTS

The design and detailing requirements for components of the seismic resisting system are stated in this section. General detailing requirements are specified in Sec. 3.7. Some of the requirements introduced here are not found in present code provisions. All are spelled out in considerably more detail and most are more stringent than those in other provisions. The major reasons for this are presented below.

The provision of detailed design ground motions and requirements for analysis of the structure do not by themselves make a building earthquake resistant. Additional design requirements are necessary to provide a consistent degree of earthquake resistance in buildings. The more severe the expected seismic ground motions, the more stringent these additional design requirements should be. Not all of the necessary design requirements are expressed in codes, and although experienced seismic design engineers account for them, engineers lacking experience in the design and construction of earthquake-resistant structures often overlook them.

Considerable uncertainties exist regarding:

- The actual dynamic characteristics of future earthquake motions expected at a building site;
- The soil-structure-foundation interaction;

- The actual response of buildings when subjected to seismic motions at their foundations; and

- The mechanical characteristics of the different structural materials, particularly when they undergo significant cyclic straining in the inelastic range that can lead to severe reversals of strains.

It should be noted that the overall inelastic response of a structure is very sensitive to the inelastic behavior of its critical regions, and this behavior is influenced, in turn, by the detailing of these regions.

Although it is possible to counteract the consequences of these uncertainties by increasing the level of design forces, it was considered more feasible to provide a building system with the largest energy dissipation consistent with the maximum tolerable deformations of nonstructural components and equipment. This energy dissipation capacity, which is usually denoted simplistically as "ductility," is extremely sensitive to the detailing. Therefore, in order to achieve such a large energy dissipation capacity, it is essential that stringent design requirements be used for detailing the structural as well as the nonstructural components and their connections or separations. Furthermore, it is necessary to have good quality control of materials and competent inspection. The importance of these factors has been clearly demonstrated by the building damage observed after both moderate and severe earthquakes.

It should be kept in mind that a building's response to seismic ground motion most often does not reflect the designer's or analyst's original conception or modeling of the structure on paper. What is reflected is the manner in which the building was constructed in the field. These provisions emphasize the importance of detailing and recognize that the detailing requirements should be related to the expected earthquake intensities and the importance of the building's function and/or the density and type of occupancy. The greater the expected intensity of earthquake ground-shaking and the more important the building function or the greater the number of occupants in the building, the more stringent the design and detailing requirements should be. In defining these requirements, the *Provisions* introduce the concept of Seismic Performance Categories (Table 1-2), which relates to the coefficient A_v (Sec. 1.4.1) and the Seismic Hazard Exposure Group (Sec. 1.4.2).

3.6.1 Seismic Performance Category A

Because of the very low seismicity associated with regions of A_v less than 0.05, it is considered appropriate for Category A buildings to require only good quality of construction materials and adequate ties and anchorage as specified in Sec. 3.7.5, 3.7.6, 3.7.7, and 7.3.

Category A buildings will be constructed in a large portion of the United States that is generally subject to strong winds but low earthquake risk. Those promulgating construction regulations for these areas may wish to consider many of the low-level seismic provisions as being suitable to reduce the windstorm risk. Since the *Provisions* consider only earthquakes, no other requirements are prescribed for Category A buildings. Only wind design in accordance with the local code and ties and wall anchorage are required by these provisions.

In low earthquake risk areas, it is unrealistic to believe that construction practices will change overnight. However, if existing requirements can be improved gradually, a major reduction in potential hazard can be achieved at low cost and with little inconvenience.

3.6.2 Seismic Performance Categories B and C

Category B and C buildings will be constructed in the largest portion of the United States. Earthquake-resistant requirements are increased appreciably over Category A requirements, but they are still quite simple compared to present requirements in areas of high seismicity. For concrete and masonry structures, the increases are taken in two steps; for steel and wood, a single step is taken since there are no differences between Categories B and C for steel and wood.

The material requirements in Chapters 9 through 12 for Category B are somewhat more restrictive than those for Category A.

The Category B and C requirements specifically recognize the need to design diaphragms, provide collector bars, and provide reinforcing around openings. These requirements may seem elementary and obvious but, because they are not specifically covered in current codes, many engineers totally neglect them. A nominal interconnection between pile caps and caissons also is required.

3.6.3 Seismic Performance Category D

Category D requirements compare roughly to present design practice in California seismic areas for buildings other than schools and hospitals. All masonry must be reinforced. All moment resisting frames of concrete or steel must meet ductility requirements. Interaction effects between structural and nonstructural elements must be investigated. Foundation interaction requirements are increased.

Experience in past earthquakes has demonstrated that unreinforced masonry or unreinforced concrete platforms perform poorly and are hazardous even when used in nonstructural elements. Consequently, all concrete and masonry construction must be reinforced for Category D construction.

3.6.4 Seismic Performance Category E

Category E construction is required for critical structures in relatively high seismic zones. It is deemed prudent that these structures not be located over the trace of an active fault that could cause ground rupture (see Sec. 1.4.4). Because of the necessity for reduced risk, height limitations are reduced (see Sec. 3.3.5). The specific material provisions include additional requirements and limitations for the design of this building category.

3.7 STRUCTURAL COMPONENT LOAD EFFECTS

This section specifies that the direction of the applied seismic force be that which produces the most critical load effect on the building. In past codes, it was necessary only to independently consider loads on the main axes of the building. For beams and girders, this gives maximum design stresses. However, if earthquake forces affect the building in a direction other than the main axes, corner columns can be subjected to higher stresses, which may partially explain the vulnerability of such columns in past earthquakes. Sec. 3.7.2 requires for Category D or E buildings that the effects from seismic loads applied in one direction be combined with those from the other direction. This may affect more than just the columns.

The second order effect that is referenced is explained more fully in Sec. 4.6.

3.7.1 Combination of Load Effects

Various combination-of-load-effects formulas and other data were reviewed before arriving at Eq. 3-1, 3-2, and 3-2a. Since 1956, for example, the American Concrete Institute (ACI) has based design on the cross-sectional strength of component members and has included load combinations that are believed to be consistent with the strength reduction factors (based in part on considerations of statistical variability of properties) to produce a margin of safety for most design loading that is generally acceptable to the design professions. No specific study was made for earthquake loading, and the load combinations were set to be compatible with previous working stress load combinations.

A subcommittee of American National Standards Institute (ANSI) Committee 58.1 also studied the problem to arrive at a compatible combination of load effects for all building system materials. Its work was stated in terms of design seismic motions ordinarily used for allowable stress design, such as by SEAOC and *UBC* prior to 1987.

Sec. 3.7.1

After carefully evaluating the available material and past experience and exercising reasonable engineering judgment, it was decided that the *Provisions* would express the load effect combinations involving seismic design in a format similar to that used in ACI 318 but with the values changed as indicated below.

The basic load factor used in ACI 318 to account for variability of dead load effects is 0.75 • 1.4 = 1.05 (the 1.4 was 1.5 before 1971). This factor combines with the appropriate understrength factor to produce a design that is judged adequate on the basis of the ultimate strength of individual members. On an average, actual dead loads have been found to be 5 to 10 percent larger than those calculated in design. Thus, it is reasonable to use a factor of 1.1 on dead load in seismic design. However, where the dead load aids in resisting the seismic load effect, the comparable load factor is 0.9.

In Eq. 3-1 and 3-2, a factor of $\mp 0.5\ A_v$ was placed on the dead load to account for the effects of vertical acceleration. The $0.5\ A_v$ factor on dead load is not intended to represent the total vertical response. The concurrent maximum response of vertical accelerations and horizontal accelerations, direct and orthogonal, is unlikely and, therefore, the direct addition of responses was not considered appropriate. For elements in which tensile mode of failure is relatively brittle, a more conservative factor of $0.7-0.5\ A_v$ on the dead load was chosen for Eq. 3-2a.

The live load factor of ACI 318 is 0.75 • 1.7 = 1.3. The factor 0.75 in the ACI equation represents the reduced likelihood of the full live load being present at the instant of the earthquake. The terms "maximum lifetime live load" and "instantaneous live load" are used. The maximum lifetime live load is assumed to be represented by the code-specified live loads. In most instances, the actual instantaneous live load is very much smaller than the maximum lifetime live load, which acts for a short time period and is generally applied to a small portion of the structure. For the purpose of the *Provisions*, it was decided to use only the code-specified loads for the present. A load factor of 1.0 was chosen to partially recognize the lower values for the instantaneous live load for combination with earthquake load effects.

For combination with the design earthquake, it is assumed that an instantaneous snow load for combination with earthquake loads is the same as that expressed in the 1976 *UBC*.

The design basis expressed in Sec. 3.1 reflects the fact that the specified earthquake loads are at the design level without amplification by load factors; thus the load factor of 1.0 is assigned to the earthquake load effects in Eq. 3-1, 3-2, and 3-2a.

3.7.2 Orthogonal Effects

Earthquake forces act in both principal directions of the building simultaneously, but the earthquake effects in the two principal directions are unlikely to reach their maximum simultaneously. This section provides a reasonable and adequate method for combining them. It requires that structural elements be designed for 100 percent of the effects of seismic forces in one principal direction combined with 30 percent of the effects of seismic forces in the orthogonal direction. The following combinations of effects of gravity loads, effects of seismic forces in the x-direction, and effects of seismic forces in the y-direction (orthogonal to x-direction) thus pertain:

gravity $\pm$ 100% of x-direction $\pm$ 30% of y-direction

gravity $\pm$ 30% of x-direction $\pm$ 100% of y-direction

The combination and signs (plus or minus) requiring the greater member strength are used for each member. Orthogonal effects are slight on beams, girders, slabs, and other horizontal elements that are essentially one-directional in their behavior, but they may be significant in columns or other vertical members that participate in resisting earthquake forces in both principal directions of the building. For two-way slabs, orthogonal effects at slab-to-column connections can be neglected provided the moment transferred in the minor direction does not exceed 30 percent of that transferred in the orthogonal direction and there is adequate reinforcement within lines one and one-half times the slab thickness either side of the column to transfer all the minor direction moment.

3.7.3 Discontinuities in Strength of Vertical Resisting System

This section requires consideration of discontinuities in strength. It is not generally recognized that large discontinuities in story strength can cause adverse response effects in a building. Usual practice is to determine what size, length, or strength of resisting elements is required; if more than the required strength is provided, so much the better. Unfortunately, the extra strength in a story, if significantly different from that in adjacent stories, can produce responses that vary greatly from those calculated by using the procedures in Chapter 4 or 5 due to the concentration of inelastic deformations in a weak story. A prohibition on weak story buildings is new with the 1988 Edition.

The early developers of the *Provisions* considered the following approach to this problem:

1. Compute the ratio of shear capacity to the design shear for each story. Denote this ratio for story n by r_n.

2. Compute, r, the average of r_n over all stories.

3. If for any story r_n is less than $2/3\ r$, modify R and C_d for the building as given by Table 3-2 to $\tilde{R}$ and $\tilde{C}_d$ where:

$$\tilde{C}_d = 1 + (C_d - 1)/2$$

and

$$\tilde{R} = (\tilde{C}_d/C_d)R.$$

4. Use $\tilde{R}$ instead of R to recompute the lateral forces, $\tilde{C}_d$ instead of C_d in computing story drifts.

It is believed that further study should be given to this problem.

3.7.4 Nonredundant Systems

Design consideration should be given to potentially adverse effects where there is a lack of redundancy. Because of the many unknowns and uncertainties in the magnitude and characteristics of earthquake loading, in the materials and systems of construction for resisting earthquake loadings and in the methods of analysis, good earthquake engineering practice has been to provide as much redundancy as possible in the seismic resisting system of buildings.

Redundancy plays an important role in determining the ability of the building to resist earthquake forces. In a structural system without redundant components, every component must remain operative to preserve the integrity of the building structure. On the other hand, in a highly redundant system, one or more redundant components may fail and still leave a structural system that retains its integrity and can continue to resist lateral forces, albeit with diminished effectiveness.

Redundancy often is accomplished by making all joints of the vertical load-carrying frame moment resisting and incorporating them into the seismic resisting system. These multiple points of resistance can prevent a catastrophic collapse due to distress or failure of a member or joint. (The overstrength characteristics of this type of frame were discussed in the commentary on Sec. 3.1.)

Redundant characteristics also can be obtained by providing several different types of seismic resisting systems in a building. The backup system can prevent catastrophic effects if distress occurs in the primary system.

In summary, it is good practice to incorporate redundancy into the seismic resisting system and not to rely on any system wherein distress in any member may cause progressive or catastrophic collapse.

3.7.5 Ties and Continuity

The analysis of a structure and the provision of a design ground motion alone do not make a structure earthquake resistant; additional design requirements are necessary to provide adequate earthquake resistance in buildings. Experienced seismic designers normally fill these requirements, but because some were not formally specified, they often were overlooked by inexperienced engineers.

Probably the most important single attribute of an earthquake-resistant building is that it is tied together to act as a unit, but this was not stated as a requirement in former provisions. This attribute not only is important in earthquake-resistant design, but also is indispensable in resisting high winds, floods, explosion, progressive failure, and even such ordinary hazards as foundation settlement. Sec. 3.7.5 requires that all parts of the building (or unit if there are separation joints) be so tied together that any part of the structure is tied to the rest to resist a force of $A_v/3$ (with a minimum of 5 percent g) times the weight of the smaller. In addition, beams must be tied to their supports or columns and columns to footings for a minimum of 5 percent of the dead and live load reaction. Furthermore, certain connections of buildings with plan irregularities must be designed for higher forces than calculated due to the simplifying assumptions used in the analysis by Chapters 3, 4, and 5.

3.7.6 Concrete or Masonry Wall Anchorage

One of the major hazards from buildings during an earthquake is the pulling away of heavy masonry or concrete walls from floors or roofs. Although requirements for the anchorage to prevent this separation are common in highly seismic areas, they have been minimal or nonexistent in most other parts of the country. This section requires that anchorage be provided in any locality to the extent of 1,000 A_v pounds per linear foot (plf). This requirement alone may not provide complete earthquake-resistant design, but observations of earthquake damage indicate that it can greatly increase the earthquake resistance of buildings and reduce hazards in those localities where earthquakes may occur but are rarely damaging.

3.7.7 Anchorage of Nonstructural Systems

Anchorage of nonstructural systems and components of buildings is required when prescribed in Chapter 8.

3.7.8 Collector Elements

Many buildings have shear walls or other bracing elements that are not uniformly spaced around the diaphragms. Such conditions require that collector or drag members be provided. A simple illustration is shown in Figure C3-6a. Consider a building as shown in the plan with four short shear walls at the corners arranged as shown. For north-south earthquake forces, the diaphragm shears on Line AB are uniformly

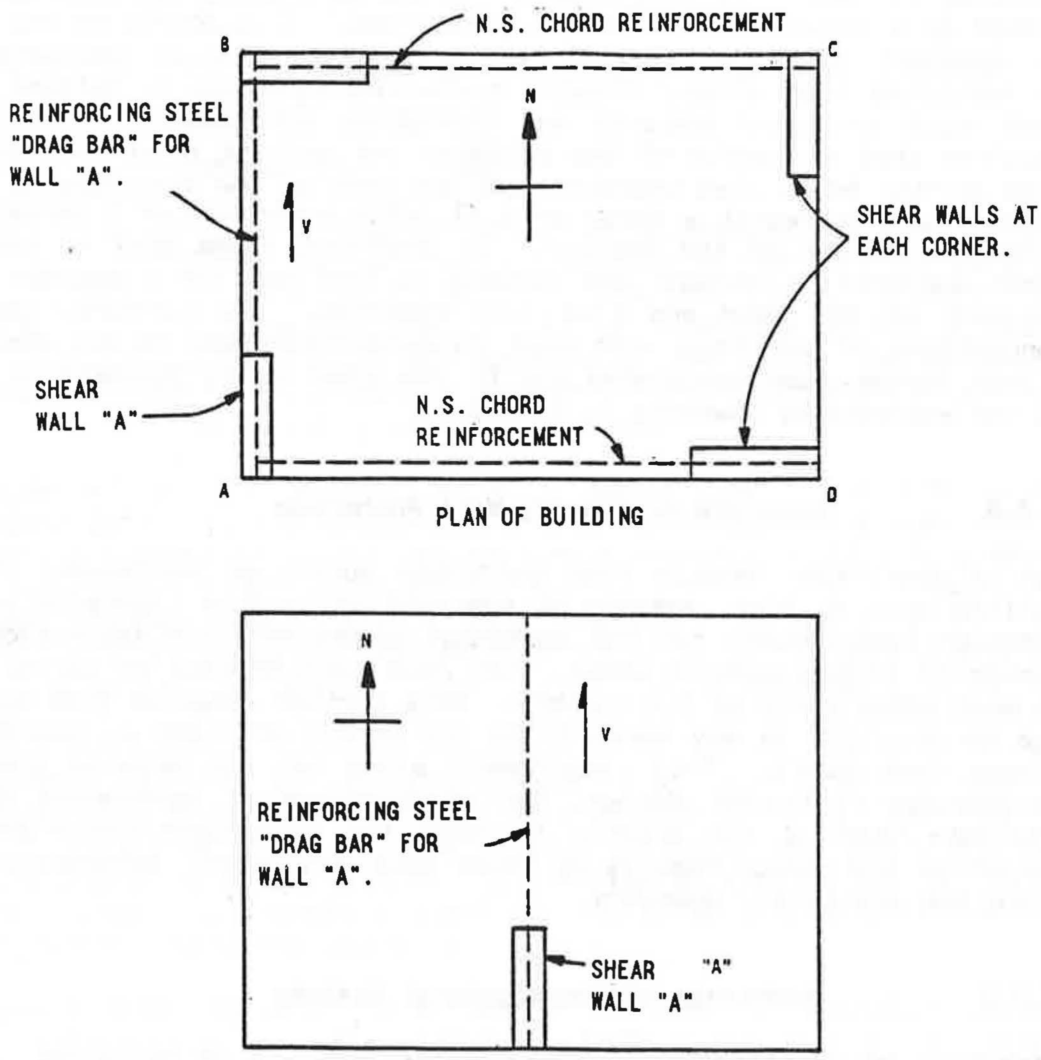


FIGURE C3-6
Collector element used to (a) transfer shears and (b) to transfer drag forces from diaphragm to shear wall.

distributed between A and B if the chord reinforcing is assumed to act on Lines BC and AD. However, wall A is quite short so reinforcing steel is required to collect these shears and transfer them to the wall. If Wall A is a quarter of the length of AB, the steel must carry, as a minimum, three-fourths of the total shear on Line AB. The same principle is true for the other walls. In Figure C3-6b reinforcing is required to collect the shears or drag the forces from the diaphragm into the shear wall. Similar collector elements are needed in most shear walls and some frames.

3.7.9 Diaphragms

Diaphragms are deep beams or trusses that distribute the lateral loads from their origin to the components where they are resisted. As such, they are subject to shears, bending moments, direct stresses (truss member, collector elements), and deformations. The deformations must be minimized in some cases because they could overstress the walls to which they are connected. The amount of deflection permitted in the diaphragm must be related to the ability of the walls (normal to the direction being analyzed) to deflect without failure.

A detail commonly overlooked by many engineers is the requirement to tie the diaphragm together so that it acts as a unit. Wall anchorages tend to tear off the edges of the diaphragm; thus, the ties must be extended into the diaphragm so as to develop adequate anchorage. During the San Fernando earthquake, seismic forces from the walls caused separations in roof diaphragms 20 or more feet from the edge in several industrial buildings.

When openings occur in shear walls, diaphragms, etc., it is not adequate to only provide temperature trimbars. The chord stresses must be provided for and the chords anchored to develop the chord stresses by embedment. The embedment must be sufficient to take the reactions without overstressing the material in any respect. Since the design basis depends on an elastic analysis, the internal force system should be compatible with both statics and the elastic deformations.

3.7.10 Bearing Walls

A minimum anchorage of bearing walls to diaphragms or other resisting elements is specified. To ensure that the walls and supporting framing system interact properly, it is required that the interconnection of dependent wall elements and connections to the framing system have sufficient ductility or rotational capacity, or strength, to stay as a unit. Large shrinkage or settlement cracks can significantly affect the desired interaction.

3.7.11 Inverted Pendulum-Type Structures

Inverted pendulum-type structures have a large portion of their mass concentrated near the top and, thus, have essentially one degree of freedom in horizontal translation. Often the structures are T-shaped with a single column supporting a beam or slab at the top. For such a structure, the lateral motion is accompanied by rotation of the horizontal element of the T due to rotation at the top of the column, resulting in vertical accelerations acting in opposite directions on the overhangs of the structure. Dynamic response amplifies this rotation; hence, a bending moment would be induced at the top of the column even though the procedures of Sec. 4.2 and 4.5 would not so indicate. A simple provision to compensate for this is specified in this section. The bending moments due to the lateral force are first calculated for the base of the column according to the provisions of Sec. 4.2 and 4.5. One-half of the calculated bending moment at the base is applied at the top and the moments along the column are varied from 1.5 M at the base to 0.5 M at the top. The addition of one-half the moment calculated at the base in accordance with Sec. 4.2 and 4.5 is based on analyses of inverted pendulums covering a wide range of practical conditions.

3.7.12 Vertical Seismic Motions for Buildings Assigned to Categories D and E

This section is intended to cover the effects of vertical ground motion where they are most important. Factors of safety provided for gravity load design, coupled with the small likelihood that maximum live loads and earthquake loads would occur simultaneously, introduce some protection against the effects of the vertical component of ground motion. Consequently, there is need for special design for vertical ground accelerations only when the effects are significant when compared with those from horizontal accelerations. Requirements for providing protection against the possible effects of the vertical component of earthquake motions are given. In the case of standard structures, these effects are taken into account by the variation of 0.5 A_v which is placed on the dead load (see Sec.3.7.1). A reduction in the gravity forces due to the response to the vertical component of ground motions can be considerably more detrimental in the case of prestressed horizontal components for similar but regularly reinforced concrete components. Thus, it is recommended that Eq. 3-2 be replaced by Eq. 3-2a. To account for the effects of vertical vibration of horizontal cantilever members, it is recommended that they be designed for a net upward force of 0.2 Q_D. The structural members most vulnerable to vertical earthquake forces are prestressed and cantilevered beams, girders, and slabs.

The specific procedures are based in part on the premise that the vertical accelerations that would develop in a building are very close to those corresponding to a structure that is perfectly rigid in the vertical direction. This is a reasonable basis provided the horizontal structural members can develop moderate ductility factors. Design requirements presented elsewhere in the *Provisions* usually will ensure such ductility capacity for downward inertia forces. To achieve it for upward inertia forces, connections in precast concrete structures and reinforcement in concrete members should be capable of resisting at least some reversal of vertical forces. This is not automatically fulfilled by simply supported or cantilevered beams, girders, and slabs or by many prestressed concrete members.

3.8 DEFLECTION AND DRIFT LIMITS

This section provides procedures for the limitation of story drift. The term "drift" has two connotations:

- "Story drift" is the maximum lateral displacement within a story (i.e., the displacement of one floor relative to the floor below caused by the effects of seismic loads).

- The lateral displacement or deflection due to design forces is the absolute displacement of any point in the structure relative to the base. This is not "story drift" and is not to be used for drift control or stability considerations since it may give a false impression of the effects in critical stories. However, it is important when considering seismic separation requirements.

There are many reasons for controlling drift; one is to control member inelastic strain. Although use of drift limitations is an imprecise and highly variable way of controlling strain, this is balanced by the current state of knowledge of what the strain limitations should be.

Stability considerations dictate that flexibility be controlled. The stability of members under elastic and inelastic deformation caused by earthquakes is a direct function of both axial loading and bending of members. A stability problem is resolved by limiting the drift on the vertical load carrying elements and the resulting secondary moment from this axial load and deflection (frequently called the P-delta effect). Under small lateral deformations, secondary stresses are normally within tolerable limits. However, larger deformations with heavy vertical loads can lead to significant secondary moments from the P-delta effects in the design. The drift limits indirectly provide upper bounds for these effects.

Buildings subjected to earthquakes need drift control to restrict damage to partitions, shaft and stair enclosures, glass, and other fragile nonstructural elements and, more importantly, to minimize differential movement demands on the seismic safety elements. Since general damage control for economic reasons is not a goal of this document and since the state of the art is not well developed in this area, the drift limits have been established without regard to considerations such as present worth of future repairs versus additional structural costs to limit drift. These are matters for building owners and designers to examine. To the extent that life might be excessively threatened, general nonstructural damage to nonstructural and seismic safety elements is a drift limit consideration.

The design story drift limits of Table 3-5 reflect consensus judgment taking into account the goals of drift control outlined above. In terms of life safety and damage control objectives, the drift limits should yield a substantial, though not absolute, measure of safety for well detailed and constructed brittle elements and provide tolerable limits wherein the seismic safety elements can successfully perform, provided they are designed and constructed in accordance with these provisions.

To provide a higher performance standard, the drift limit for the essential facilities of Seismic Hazard Exposure Group III is more stringent than the limit for Groups I and II.

The drift limits for low-rise structures are relaxed somewhat provided the finishes are not brittle. The type of steel building envisioned by the exception to the table would be similar to a prefabricated steel structure with metal skin. When the more liberal drift limits are used, it is recommended that special provisions be provided for the seismic safety elements to accommodate the drift.

It should be emphasized that the drift limits, Δ_a, of Table 3-5 are story drifts and, therefore, are applicable to each story (i.e., they must not be exceeded in any story even though the drift in other stories may be well below the limit.) The limit, Δ_a is to be compared to the design story drift as determined by Sec. 4.6.1.

Stress or strength limitations imposed by design level forces occasionally may provide adequate drift control. However, it is expected that the design of moment resisting frames, especially steel building frames, and the design of tall, narrow shear wall or braced frame buildings will be governed at least in part by drift considerations. In areas having a large seismic coefficient, A_v, it is expected that seismic drift considerations will predominate for buildings of medium height. In areas having a low seismic coefficient and for very tall buildings in areas with large coefficients, wind considerations may generally will control, at least in the lower stories.

Due to probable first mode drift contributions and C_s being generally conservative at higher values of T or T_a, the Chapter 4 ELF Procedure may be too conservative for drift design of very tall moment-frame buildings. It is suggested for these buildings, where the first mode would be responding in the displacement region of a response spectra (where displacements would be essentially independent of stiffness), that the Modal Analysis Procedure of Chapter 5 be used for design even when not required by Sec. 3.5.

Building separations and seismic joints are separations between two adjoining buildings or parts of the same building, with or without frangible closures, for the purpose of permitting the adjoining buildings or parts to respond independently to earthquake ground motion. Unless all portions of the structure have been designed and constructed to act as a unit, they must be separated by seismic joints. For irregular structures that cannot be expected to act reliably as a unit, seismic joints should be utilized to separate the building into units whose independent response to earthquake ground motion can be predicted.

Although the *Provisions* do not give precise formulations for the separations, it is required that the distance be "sufficient to avoid damaging contact under total deflection" in order to avoid interference and possible destructive hammering between buildings. It is recommended that the distance be equal to the total of the lateral deflections of the two units assumed deflecting toward each other (this involves increasing separations with height). If the effects of hammering can be shown not to be detrimental, these distances can be reduced. For very rigid shear wall structures with rigid diaphragms whose lateral deflections cannot be reasonably estimated, it is suggested that older code requirements for structural separations of at least 1 inch plus 1/2 inch for each 10 feet of height above 20 feet be followed.

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Chapter 4 Commentary

EQUIVALENT LATERAL FORCE PROCEDURE

4.1 GENERAL

This chapter discusses the Equivalent Lateral Force (ELF) Procedure for seismic analysis of buildings.

4.2 SEISMIC BASE SHEAR

The heart of the ELF procedure is Eq. 4-1 for base shear, which gives the total seismic design force, V, in terms of two factors: a seismic coefficient, C_s, and the total gravity load of the building, W. Equations 4-2 and 4-3 give the coefficient C_s, which defines the design spectrum. This spectrum is discussed more fully in Sec. 1.4.1 of the "Chapter 1 Commentary."

The gravity load W is the total weight of the building and that part of the service load that might reasonably be expected to be attached to the building at the time of an earthquake. It includes partitions, permanent or movable, plus permanent equipment such as mechanical and electrical equipment, piping, and ceilings. The normal human live load is taken to be negligibly small in its contribution to the seismic lateral forces. Buildings designed for storage or warehouse usage should have at least 25 percent of the design floor live load included in the weight, W. Snow loads up to 30 psf are not considered (see Sec. 2.1). Freshly fallen snow would have little effect on the lateral force in an earthquake; however, ice loading would be more or less firmly attached to the roof of the building and would contribute significantly to the inertia force. For this reason, the effective snow load is taken as the full snow load for those regions where the snow load exceeds 30 psf with the proviso that the local Regulatory Agency may allow the snow load to be reduced up to 80 percent. The question of how much snow load should be included in W is really a question of how much ice buildup or snow entrapment can be expected for the roof configuration or site topography, and this is a question best left to the discretion of the local Regulatory Agency.

The seismic coefficient formula and the various factors contained therein were arrived at as explained below.

Elastic Acceleration Response Spectra

See Sec. 1.4.1 of the "Chapter 1 Commentary" for a full discussion of the shape of the spectra accounting for dynamic response amplification and the effect of site response, or the Soil Profile factor.

Elastic Design Spectra

As described in Sec. 1.4.1, the elastic acceleration response spectra for earthquake motions has a descending branch for longer values of T, the period of vibration of the system, and it varies roughly as 1/T. However, because of a number of reasons associated with the structural behavior of long-period buildings, it was decided that ordinates of design spectra should not decrease as rapidly with T; hence, the period T appears to the two-third power in the denominator of Eq. 4-2. Among the reasons for designing long-period buildings more conservatively are the following:

- The fundamental period of a building increases with number of stories. The longer the T, the larger the likely number of stories and the number of degrees of freedom and, hence, the more likely that high ductility requirements can be concentrated in a few stories of the building, at least for some earthquakes.

- The number of potential failure modes increases generally with T. If design spectra were proportional to response spectra for single-degree-of-freedom systems, the probability of failure would increase with T.

- Building instability is more of a problem with increasing T.

Estimated Period

In the denominator of Eq. 4-2, T is intended to be an estimate of the fundamental period of vibration of the building. Methods of mechanics cannot be employed to calculate the vibration period before a building design, at least a preliminary one, is available. Simple formulas that involve only a general description of the building type (e.g., steel moment frame, concrete moment frame, shear wall system, braced frame), and overall dimensions (e.g. height and plan length) are therefore necessary to estimate the vibration period in order to calculate an initial base shear and proceed with a preliminary design. For preliminary member sizing, it is advisable that this base shear and the corresponding value of T be conservative. Even for final design, use of a large value for T is unconservative. Thus, the value of T *used in design* should be smaller than the true period of the building. Equations 4-4 and 4-5 for the approximate period T_a are therefore intended to provide conservative estimates of the fundamental period of vibration. An upper bound is placed on T based on T_a and the factor C_a.

The coefficient C_a accommodates the probable fact that buildings in areas with lower lateral force requirements probably will be more flexible. Furthermore, it results in less dramatic changes from present practice in lower risk areas. It is generally accepted that the *empirical* equations for T_a are tailored to fit the type of construction common in areas with high lateral force requirements.

It is unlikely that buildings in lower risk seismic areas would be designed to produce as high a drift level as allowed in the provisions due to stability problems (P-delta) and wind requirements. For buildings that are actually "controlled" by wind, the calculation of a large T will not really result in a lower design force; thus, use of this approach in high-wind regions should not result in unsafe design.

Taking the seismic base shear coefficient to vary as $1/T^{2/3}$ and assuming that the lateral forces are distributed linearly over the height and the deflections are controlled by drift limitations, a simple analysis of the vibration period by Rayleigh's method (Clough and Penzien, 1975; Newmark and Rosenblueth, 1971; Thomson, 1965; Wiegel, 1970) leads to the conclusion that the vibration period of moment resisting frame structures varies roughly as $h_n^{3/4}$ where h_n equals the total height of the building as defined elsewhere. Equation 4-4 is therefore appropriate and the values of the coefficient C_T have been established to produce values for T_a generally lower than the true fundamental vibration period of moment frame buildings. This is apparent in Figures C4-1 and C4-2, wherein Eq. 4-4 is compared with fundamental vibration periods computed from accelerograph records from upper stories of several buildings during the 1971 San Fernando earthquake. The optional use of $T = 0.1N$, an approximation long in use, was restored in the 1988 Edition for low to moderate height frames.

Equation 4-5 is identical to the formula used for many years in SEAOC recommendations. It is apparent from Figure C4-3 that this would generally underestimate the fundamental vibration period of reinforced-concrete shear-wall buildings. Equation 4-5 is to be used for all buildings other than those included in Figures C4-1 and 4-2 because there is insufficient data on measured periods of such building types and materials to permit development of special formulas. It is expected to provide underestimates of periods of vibration for other building types.

As an exception to Eq. 4-4 and 4-5, these provisions allow the calculated fundamental period of vibration, T, of the seismic resisting system to be used in calculating the base shear. However, the period, T, used may not exceed C_aT_a with T_a determined from Eq. 4-4 or 4-5 as appropriate.

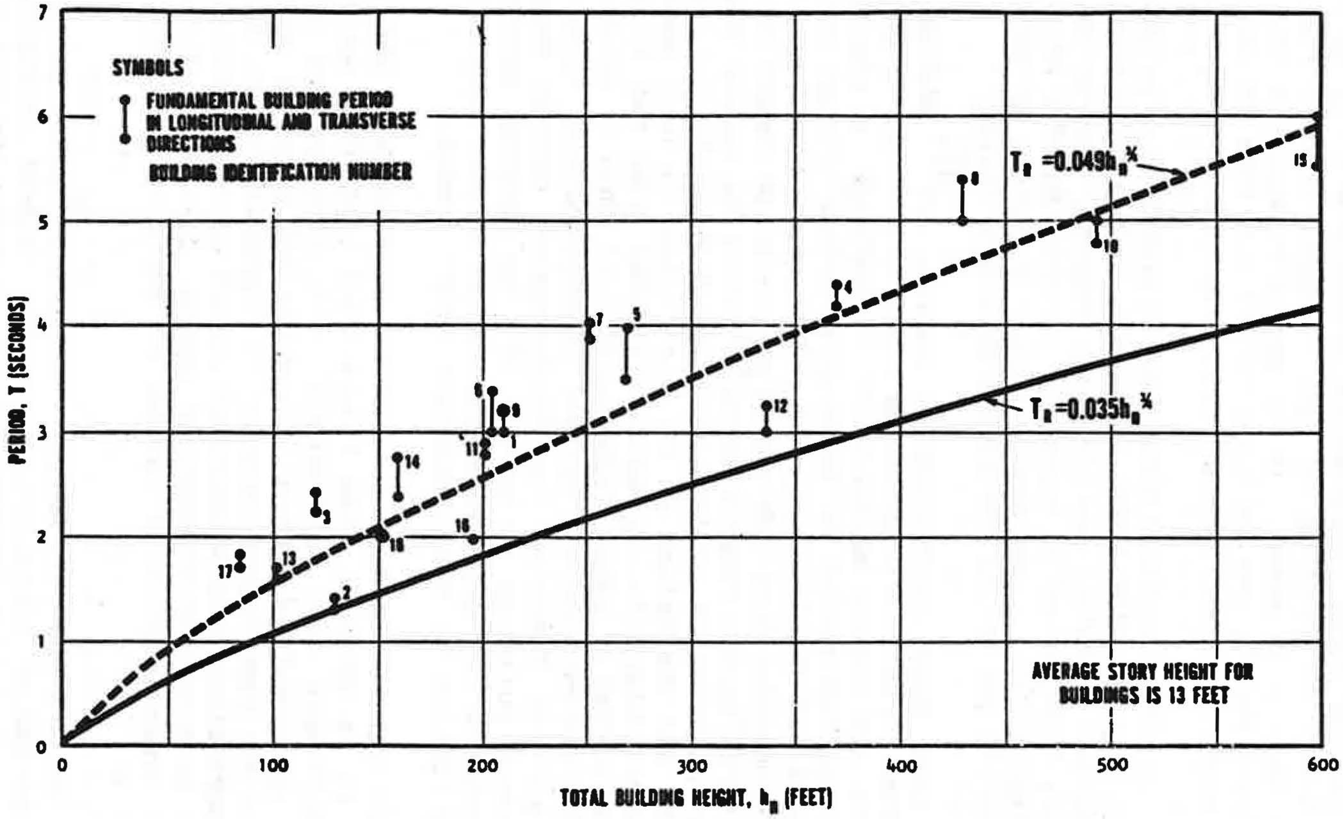

FIGURE C4-1

Periods computed from accelerograph records during the 1971 San Fernando earthquake--steel frames. Eq 4-4 ($0.035\ h_n^{3/4}$) is intended to be a conservative estimate. The mean value estimate is $0.049\ h_n^{3/4}$.

The identification numbers, names, and addresses of the buildings considered are as follows: (1) KB Valley Center, 15910 Ventura; (2) Jet Propulsion Lab Administration Building No. 180; (3) 6464 Sunset Boulevard; (4) 1900 Avenue of the Stars, Century City; (5) 1901 Avenue of the Stars, Century City; (6) 1880 Century Park East, Century City; (7) 1888 Century Park East Office Tower, Century City; (8) Mutual Benefit Life Plaza, 5900 Wilshire Boulevard; (9) Department of Water and Power, 111 North Hope Street; (10) Union Bank Building, 445 South Figueroa; (11) Kajima International, 250 East First Street; (12) Bunker Hill Tower, 800 West First Street; (13) 3407 West Sixth Street; (14) Occidental Building, 1150 South Hill Street; (15) Crocker Citizens Bank Building, 611 West Sixth Street; (16) Sears Headquarters, 900 South Fremont, Alhambra; (17) 5260 Century Boulevard.

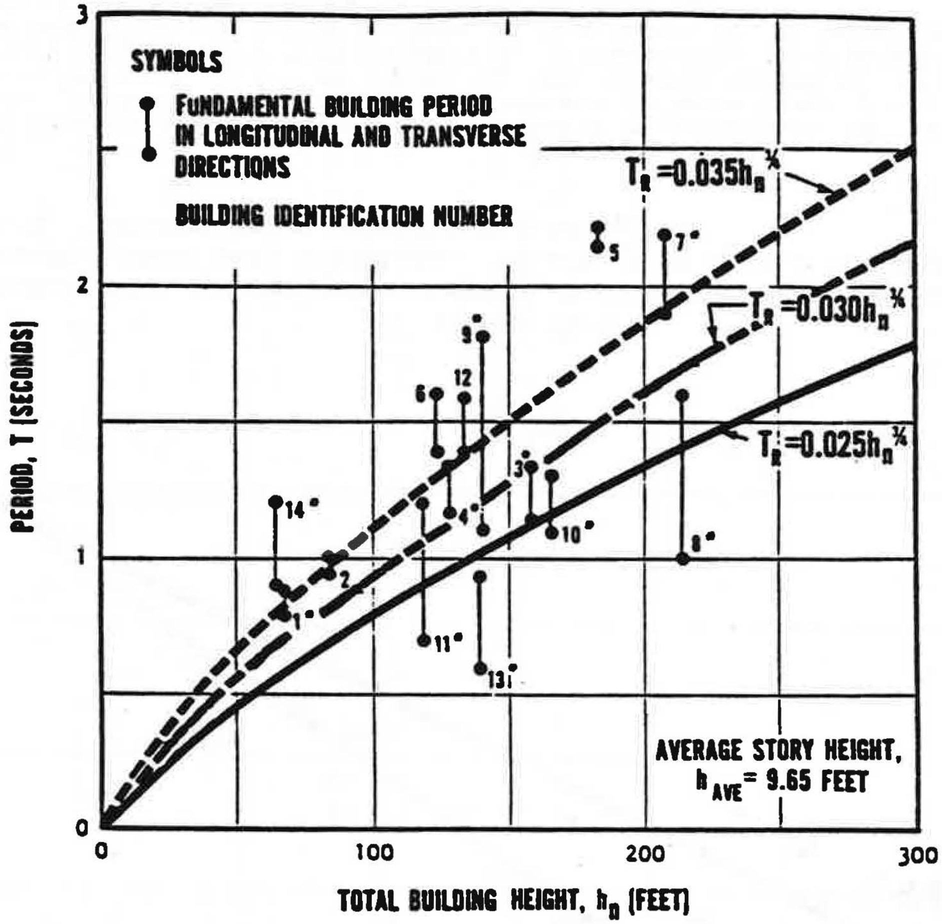

FIGURE C4-2

Periods computed from accelerograph records during the 1971 San Fernando earthquake--reinforced concrete frames. Eq 4-4 (0.030 $h_n^{3/4}$) is intended to be a conservative estimate. The mean value estimate is 0.035 $h_n^{3/4}$. Prior to the 1988 Edition, 0.025 $h_n^{3/4}$ was the estimate used as the conservative estimate.

The identification numbers, names, and addresses of the buildings considered are as follows: (1) Holiday Inn, 8244 Orion Street; (2) Valley Presbyterian Hospital, 15107 Vanowen Boulevard; (3) Bank of California, 15250 Ventura Boulevard; (4) Hilton Hotel, 15433 Ventura Boulevard; (5) Sheraton-Universal, 3838 Lankership Boulevard; (6) Muir Medical Center, 7080 Hollywood Boulevard; (7) Holiday Inn, 1760 North Orchid; (8) 1800 Century Park East, Century City; (9) Wilshire Christian Towers, 616 South Normandie Avenue; (10) Wilshire Square One, 3345 Wilshire Boulevard; (11) 533 South Fremont; (12) Mohn Olympic, 1625 Olympic Boulevard; (13) 120 Robertson; (14) Holiday Inn, 1640 Marengo. Incomplete study data have suggested that Buildings 1, 3, 4, 7, 8, 9, 10, 11, 13, and 14, may not act as true frames; these building numbers are marked with an asterisk (*).

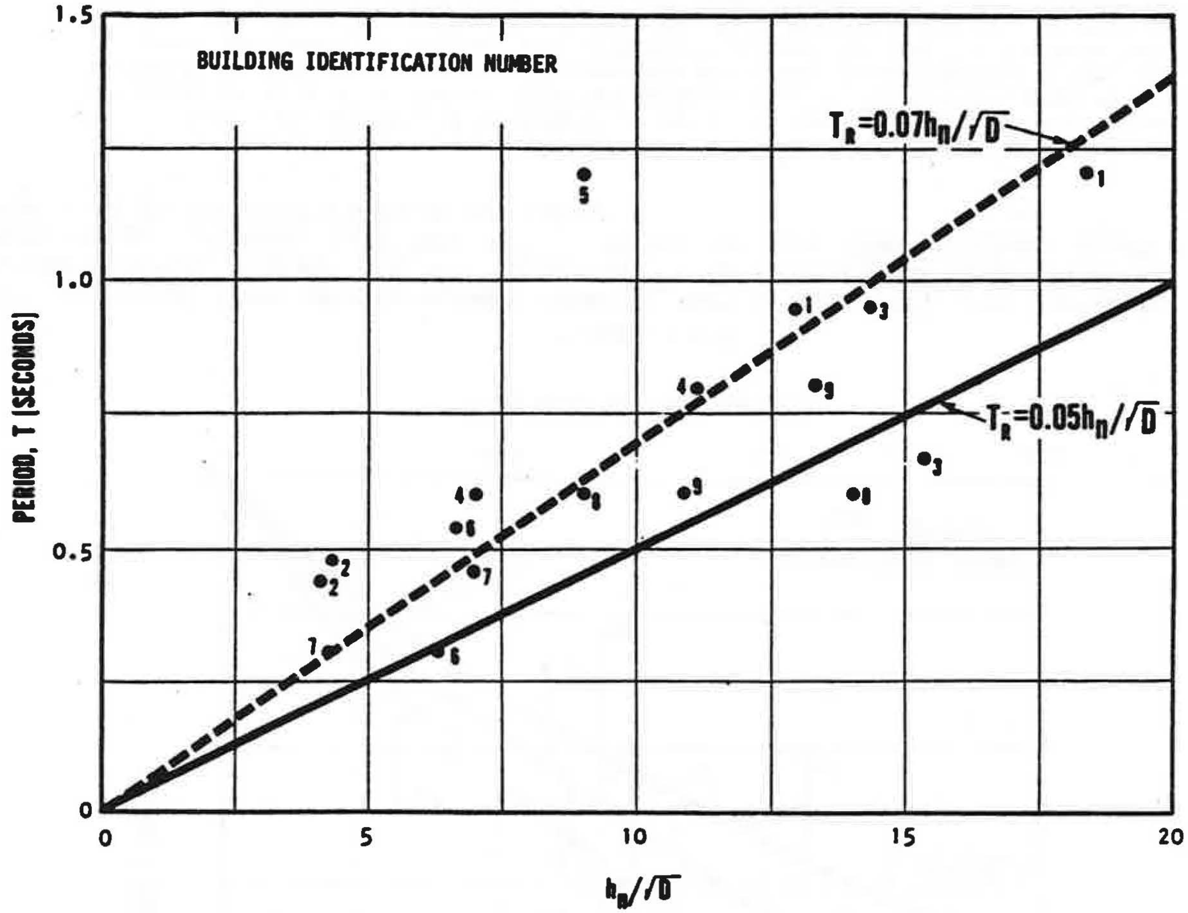

FIGURE C4-3

Periods computed from accelerograph records during the 1971 San Fernando earthquake--reinforced concrete shear wall buildings. Eq 4-5 ($0.05\ h_n/\sqrt{L}$) is intended to be a conservative estimate. The mean value estimate is $0.07\ h_n/\sqrt{L}$.

The identification numbers, names, and addresses of the buildings considered are as follows: (1) Certified Life, 14724 Ventura Boulevard; (2) Kaiser Foundation Hospital, 4867 Sunset Boulevard; (3) Millikan Library, Cal Tech, Pasadena; (4) 1888 Century Park East, Century City; (5) 3470 Wilshire Boulevard; (6) Los Angeles Athletic Club Parking Structure, 646 South Olive; (7) Parking Structure, 808 South Olive; (8) USC Medical Center, 2011 Zonal; (9) Airport Marina Hotel, 8639 Lincoln, Marina Del Ray.

For exceptionally stiff or light buildings, the calculated T for the seismic resisting system may be significantly shorter than T_a calculated by Eq. 4-4 or 4-5. For such buildings, it is recommended that the period value T be used in lieu of T_a for calculating the base shear coefficient, C_s.

The fundamental period of vibration of the seismic resisting system is to be calculated according to established methods of mechanics (Clough and Penzien, 1975; Newmark and Rosenblueth, 1971; Thomson, 1965; Wiegel, 1970). Computer programs are available for such calculations. One method of calculating the period, probably as convenient as any, is the use of the following formula based on Rayleigh's method (Clough and Penzien, 1975; Newmark and Rosenblueth, 1971; Thomson, 1965; Wiegel, 1970):

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} F_i \delta_i}} \qquad \text{(C4-1)}$$

in which F_i is the seismic lateral force at level i, w_i is the gravity load assigned in level i, δ_i is the static lateral displacement at level i due to the forces F_i computed on a linear elastic basis, and g is the acceleration of gravity.

The calculated period increases with an increase in flexibility of the structure because the δ term in the Rayleigh formula appears to the second power in the numerator but to only the first power in the denominator. Thus, if one ignores the contribution of nonstructural elements to the stiffness of the structure in calculating the deflections δ, the deflections are exaggerated and the calculated period is lengthened, leading to a decrease in the coefficient C_s and, therefore, a decrease in the design force. Nonstructural elements do not know that they are nonstructural. They participate in the behavior of the structure even though the designer may not rely on them for contributing any strength or stiffness to the structure. To ignore them in calculating the period is to err on the unconservative side. The limitation of C_aT_a is imposed as a safeguard.

Response Modification Factor

The factor R in the denominator of Eq. 4-2 is an empirical response reduction factor intended to account for both damping and the ductility inherent in the structural system at displacements great enough to surpass initial yield and approach the ultimate load displacement of the structural system. Thus, for a lightly damped building structure of brittle material that would be unable to tolerate any appreciable deformation beyond the elastic range, the factor R would be close to 1 (i.e., no reduction from the linear elastic response would be allowed). At the other extreme, a heavily damped building structure with a very ductile structural system would be able to withstand deformations considerably in excess of initial yield and would, therefore, justify the assignment of a larger response reduction factor R. Table 3-2 in the *Provisions* stipulates R coefficients for different types of building systems using several different structural materials. The coefficient R ranges in value from a minimum of 1-1/4 for an unreinforced masonry bearing wall system to a maximum of 8 for a Special Moment Frame system. The basis for the R-factor values specified in Table 3-2 was explained in the "Chapter 3 Commentary."

In establishing Eq. 4-1 for determining the design base shear of a building, the use of a factor (such as an occupancy factor) related to the Seismic Hazard Exposure Group was discussed. After lengthy consideration it was decided that arbitrarily increasing the seismic base shear is generally ineffective in improving building safety. Good connections and construction details, quality assurance procedures, and limitations on building deformation or drift will significantly improve the capability for maintenance of function and safety in critical facilities and those with a high-density occupancy. Accordingly, after comparing the design effects resulting from the preliminary version of the *Provisions* (ATC 3-06) with previous design codes, it was decided that the specified force levels provide an adequate force function for design of all buildings. However, to improve the capability for meeting the more restrictive requirements for higher Seismic Hazard Exposure Group buildings, building design categories were specified and appropriate special detailing requirements added. The reduction in the damage potential of critical facilities (Group III) was handled by using more conservative drift controls (Sec. 3.8) and by providing special design and detailing requirements (Sec. 3.6) and materials limitations (Chapters 9 through 12).

4.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

The distribution of lateral forces over the height of a building is generally quite complex because these forces are the result of superposition of a number of natural modes of vibration. The relative contributions of these vibration modes to the total forces depends on a number of factors including the shape of the earthquake response spectrum, the natural periods of vibration of the building, and the shapes of vibration modes that, in turn, depend on the mass and stiffness over the height (see Sec. 3.4). The basis of this method is discussed below. In buildings having only minor irregularity of mass or stiffness over the height, the accuracy of the lateral force distribution as given by Eq. 4-6a is much improved by the procedure described in the last portion of Sec. 3.5 of the Chapter 3 Commentary.

The lateral force at each level, x, (Figure C4-4) due to response in the first (fundamental) natural mode of vibration is:

$$f_{x1} = V_1 \frac{w_x \phi_{x1}}{\sum_{i=1}^{n} w_i \phi_{i1}} \qquad \text{(C4-2)}$$

where V_1 is the contribution of this mode to the base shear, w_i is the weight lumped at the ith level, and ϕ_i is the amplitude of the first mode at the i^{th} level. This is the same as Eq. 5-4 and 5-4a in Chapter 5 of the *Provisions*, but it is specialized for the first mode. If V_1 is replaced by the total base shear, V, these equations become identical to Eq. 4-6 and 4-6a with k = 1 if the first mode shape is a straight line and with k = 2 if the first mode shape is a parabola with its vertex at the base.

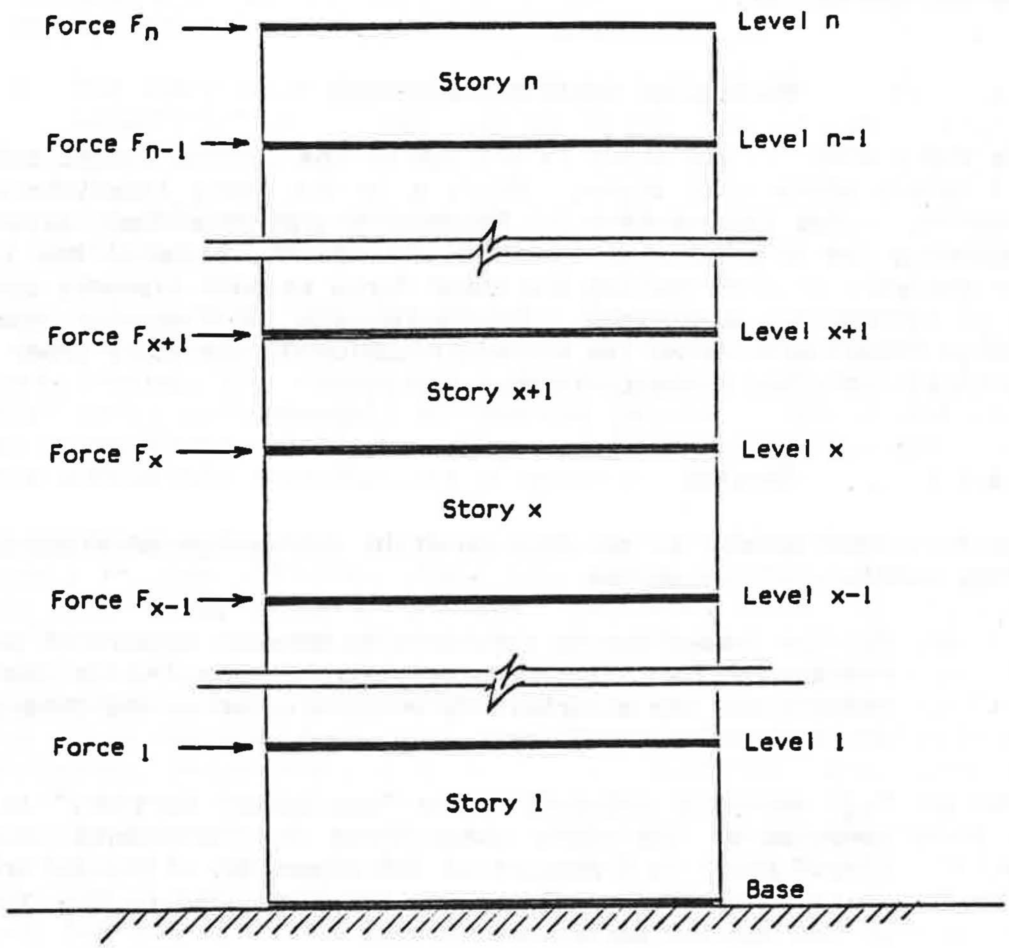

FIGURE C4-4
Description of story and level. The shear at Story x (V_x) is the sum of all the lateral forces at and above Story x (F_x through F_n).

It is well known that the influence of modes of vibration higher than the fundamental mode is small in the earthquake response of short-period buildings and that, in regular buildings, the fundamental vibration mode departs little from a straight line. This, along with the matters discussed above, provides the basis for Eq. 4-6a with $k = 1$ for buildings having a fundamental vibration period of 0.5 seconds or less.

It has been demonstrated that although the earthquake response of long-period buildings is primarily due to the fundamental natural mode of vibration, the influence of higher modes of vibration can be significant and, in regular buildings, the fundamental vibration mode lies approximately between a straight line and a parabola with the vertex at the base. Thus, Eq. 4-6a with $k = 2$ is appropriate for buildings having a fundamental period of vibration of 2.5 seconds or longer. Linear variation of k between 1 at a 0.5 second period and 2 at a 2.5 seconds period provides the simplest possible transition between the two extreme values.

4.4 HORIZONTAL SHEAR DISTRIBUTION

The story shear in any story is the sum of the lateral forces acting at all levels above that story. Story x is the story immediately below Level x. (See Figure C4-4.) Reasonable and consistent assumptions regarding the stiffness of concrete and masonry elements may be used for analysis in distributing the shear force to such elements connected by a horizontal diaphragm. Similarly, the stiffness of moment or braced frames will establish the distribution of the story shear to the vertical resisting elements in that story.

4.4.1 Torsion

The torsional moment to be considered in the design of elements in a story consists of two parts:

- M_t, the moment due to eccentricity between centers of mass and resistance for that story, is to be computed as the story shear times the eccentricity perpendicular to the direction of applied earthquake forces.

- M_{ta}, commonly referred to as "accidental torsion," is to be computed as the story shear times the "accidental eccentricity," equal to 5 percent of the dimension of the building, in the story under consideration perpendicular to the direction of the applied earthquake forces.

Computation of M_{ta} in this manner is equivalent to the procedure in Sec. 4.4 which implies that the dimension of the building is the dimension in the story where the torsional moment is being computed and that all the masses above that story should be assumed to be displaced in the same direction at one time (e.g., first, all of them to the left and, then, to the right).

Dynamic analyses assuming linear behavior indicate that the torsional moment due to eccentricity between centers of mass and resistance may significantly exceed M_t (Newmark and Rosenblueth, 1971). However, such dynamic magnification is not included in the *Provisions*, partly because its significance is not well understood for buildings designed to deform well beyond the range of linear behavior.

The torsional moment M_t calculated in accordance with this provision would be zero in those stories where centers of mass and resistance coincide. However, during vibration of the building, torsional moments would be induced in such stories due to eccentricities between centers of mass and resistance in other stories. To account for such effects, it is recommended that the torsional moment in any story be not smaller than the following two values (Newmark and Rosenblueth, 1971):

- The story shear times one-half of the maximum of the computed eccentricities in all stories below the one being analyzed and

- One-half of the maximum of the computed torsional moments for all stories above.

Accidental torsion is intended to cover the effects of several factors that have not been explicitly considered in the *Provisions*. These factors include the rotational component of ground motion about a vertical axis; unforeseeable differences between computed and actual values of stiffness, yield strengths, and dead-load masses; and unforeseeable unfavorable distributions of dead- and live-load masses.

There are indications that the 5 percent accidental eccentricity may be too small in some buildings since they may develop torsional dynamic instability. Some examples are the upper stories of tall buildings having little or no nominal eccentricity, those structures where the calculations of relative stiffnesses of various elements are particularly uncertain (e.g., those that depend largely on masonry walls for lateral force resistance or those that depend on vertical elements made of different materials), and nominally symmetrical structures that utilize core elements alone for seismic resistance or that behave essentially like elastic nonlinear systems (e.g., some prestressed concrete frames). The amplification factor for torsionally irregular buildings (Eq. 4-8) was introduced in the 1988 Edition as an attempt to account for some of these problems in a controlled and rational way. Raising the ratio of the power of two was done in recognition of the magnification effects on torsion that occur with changes in stiffness of resisting elements.

Sec. 4.4.1

The way in which the story shears and the effects of torsional moments are distributed to the vertical elements of the seismic resisting system depends on the stiffness of the diaphragms relative to vertical elements of the system.

Where the diaphragm stiffness in its own plane is sufficiently high relative to the stiffness of the vertical components of the system, the diaphragm may be assumed to be indefinitely rigid for purposes of this section. Then, in accordance with compatibility and equilibrium requirements, the shear in any story is to be distributed among the vertical components in proportion to their contributions to the lateral stiffness of the story while the story torsional moment produces additional shears in these components that are proportional to their contributions to the torsional stiffness of the story about its center of resistance. This contribution of any component is the product of its lateral stiffness and the square of its distance to the center of resistance of the story. Alternatively, the story shears and torsional moments may be distributed on the basis of a three-dimensional analysis of the structure, consistent with the assumption of linear behavior.

Where the diaphragm in its own plane is very flexible relative to the vertical components, each vertical component acts almost independently of the rest. The story shear should be distributed to the vertical components considering these to be rigid supports. Analysis of the diaphragm acting as a continuous horizontal beam or truss on rigid supports leads to the distribution of shears. Because the properties of the beam or truss may not be accurately computed, the shears in vertical elements should not be taken to be less than those based on "tributary areas." Accidental torsion may be accounted for by adjusting the position of the horizontal force with respect to the supporting vertical elements.

There are some common situations where it is obvious that the diaphragm can be assumed to be either rigid or very flexible in its own plane for purposes of distributing story shear and considering torsional moments. For example, a solid monolithic reinforced concrete slab, square or nearly square in plan, in a building with slender moment resisting frames may be regarded as rigid. A large plywood diaphragm with widely spaced and long, low masonry walls may be regarded as very flexible. In intermediate situations, the design forces should be based on an analysis that explicitly considers diaphragm deformations and satisfies equilibrium and compatibility requirements. Alternatively, the design forces should be the envelope of the two sets of forces resulting from both extreme assumptions regarding the diaphragms--rigid or very flexible.

Where the horizontal diaphragm is not continuous, the story shear can be distributed to the vertical components based on their tributary areas.

4.5 OVERTURNING

This section requires that the building be designed to resist overturning moments statically consistent with the design story shears, except for reduction factor κ in Eq. 4-9. There are several reasons for reducing the statically computed overturning moments:

- The distribution of design story shears over height computed from the lateral forces of Sec. 4.2 is intended to provide an envelope since the shears in all stories do not attain their maximum simultaneously. Thus, the overturning moments computed statically from the envelope of story shears will be overestimated.

- It is intended that the design shear envelope, which is based on the simple distribution of forces specified in Sec. 4.3, be conservative. If the shear in a specific story is close to the exact value, the shears in almost all other stories are almost necessarily overestimated. Hence, the overturning moments statically consistent with the design story shears will be overestimated.

- 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 seismic forces and, consequently, in the overturning moments.

The overturning moments computed statically from the envelope of story shears may be reduced by no more than 20 percent. This value is similar to those obtained from results of dynamic analysis taking into account for the first two reasons presented above. No reduction is permitted in the uppermost 10 stories primarily because the statically computed overturning moment in these stories may err on the unsafe side (Newmark and Rosenblueth, 1971). In any case, there is hardly any benefit in reducing the overturning moments in the stories near the top of buildings because design of vertical elements in these stories is rarely governed by overturning moments. For the eleventh to the twentieth stories from the top, linear variation of κ provides the simplest transition between the minimum and maximum values of 0.8 and 1.0.

In the design of the foundation, the overturning moment may be calculated at the foundation-soil interface using Eq. 4-9 with $\kappa = 0.75$ for all building heights. This is appropriate because a slight uplifting of one edge of the foundation during vibration leads to reduction in the overturning moment and because such behavior does not normally cause structural distress.

Formerly many building codes and design recommendations, including the 1968 SEAOC recommendations, allowed more drastic reduction in overturning moments relative to their value statically consistent with the design 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 due primarily to effects of overturning moment. In later versions of the SEAOC recommendations (1973), no reduction was allowed. The moderate reduction permitted in Sec. 4.5, which is consistent with results of dynamic analyses (Newmark and Rosenblueth, 1971), is more appropriate because use of the full statically determined overturning moment cannot be justified in light of the reasons mentioned above.

4.6 DRIFT DETERMINATION AND P-DELTA EFFECTS

This section defines the design story drift as the difference of the deflections, δ_x, at the top and bottom of the story under consideration. The deflections, δ_x, are determined by multiplying the deflections, δ_{xe} (determined from an elastic analysis), by the deflection amplification factor, C_d, given in Table 3-2. The elastic analysis is to be made for the seismic resisting system using the prescribed seismic design forces and considering the building to be fixed at the base. Stiffnesses other than those of the seismic resisting system should not be included since they may not be reliable at higher inelastic strain levels.

The deflections are to be determined by combining the effects of joint rotation of members, shear deformations between floors, the axial deformations of the overall lateral resisting elements, and the shear and flexural deformations of shear walls and braced frames. The deflections are determined initially on the basis of the distribution of lateral forces stipulated in Sec. 4.3. For frame structures, the axial deformations from bending effects, although contributing to the overall building distortion, may or may not affect the story-to-story drift; however, they are to be considered. Centerline dimensions between the frame elements often are used for analysis, but clear span dimensions with consideration of joint panel zone deformation also may be used.

For determining compliance with the story drift limitation of Sec. 3.8, the deflections, δ_x, may be calculated as indicated above for the seismic resisting system and design forces corresponding to the fundamental period of the building, T (calculated without the limit $T \leq C_a T_a$ specified in Sec. 4.2.2), may be used. The same model of the seismic resisting system used in determining the deflections must be used for determining T. The waiver does not pertain to the calculation of drifts for determining P-delta effects on member forces, overturning moments, etc. If the P-delta effects determined in Sec. 4.6.2 are significant, the design story drift must be increased by the resulting incremental factor.

The P-delta effects in a given story are due to the eccentricity of the gravity load above that story. If the story drift due to the lateral forces prescribed in Sec. 4.3 were Δ, the bending moments in the story would be augmented by an amount equal to Δ times the gravity load above the story. The ratio of the P-delta moment to the lateral force story moment is designated as a stability coefficient, θ, in Eq. 4-11. If the stability coefficient θ is less than 0.10 for every story, the P-delta effects on story shears and moments and member forces may be ignored. If, however, the stability coefficient θ exceeds 0.10 for any story, the P-delta effects on story drifts, shears, member forces, etc., for the whole building must be determined by a rational analysis.

An acceptable P-delta analysis, based upon elastic stability theory, is as follows:

1. Compute for each story the P-delta amplification factor, $a_d = \theta/(1-\theta)$. a_d takes into account the multiplier effect due to the initial story drift leading to another increment of drift that would lead to yet another increment, etc. Thus, both the effective shear in the story and the computed eccentricity would be augmented by a factor $1 + \theta + \theta^2 + \theta^3 \ldots$, which is $1/(1-\theta)$ or $(1 + a_d)$.

2. Multiply the story shear, V_x, in each story by the factor $(1 + a_d)$ for that story and recompute the story shears, overturning moments, and other seismic force effects corresponding to these augmented story shears.

This procedure effectively checks the static stability of a structure based upon its initial elastic stiffness. Some have argued that a better estimate of the stability would be found using the secant stiffness, which can be implemented by deleting the term C_d from the denominator of Eq. 4-11. Both approaches are rooted in static stability considerations. The real problem of dynamic stability is complex. The procedure in the *Provisions* may not be conservative, and it is recommended that designs producing a value of θ exceeding 0.25 be examined very carefully, particularly if the structure resembles an inverted pendulum.

Any of a number of rational analyses could be used. Some published computer programs take P-delta effects into account.

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Chapter 5 Commentary

MODAL ANALYSIS PROCEDURE

5.1-5.2 GENERAL and MODELING

Modal analysis (Newmark and Rosenblueth, 1971; Clough and Penzien, 1975; Thomson, 1965; Wiegel, 1970) generally is applicable for calculating the linear response of complex, multi-degree-of-freedom structures and is based on the fact that the response is the superposition of the responses of individual natural modes of vibration, each mode responding with its own particular pattern of deformation, the mode shape, with its own frequency, the modal frequency, and with its own modal damping. The response of the structure therefore can be modeled by the response of a number of single-degree-of-freedom oscillators with properties chosen to be representative of the mode and the degree to which the mode is excited by the earthquake motion. For certain types of damping, this representation is mathematically exact and, for building structures, numerous full-scale tests and analyses of earthquake response of structures have shown that the use of modal analysis, with viscously damped single-degree-of-freedom oscillators describing the response of the structural modes, is an accurate approximation for analysis of linear response.

Modal analysis is useful in design. The formulas describing seismic coefficients (e.g., Eq. 4-2) are simply expansions of acceleration design spectra and therefore can be used to determine the maximum response of each mode of a complete building. This maximum modal response can be expressed in several ways. For the *Provisions*, it was decided that the modal forces and their distributions over the structure should be given primary emphasis to highlight the similarity to the equivalent static methods traditional in building codes (the SEAOC recommendations and the *UBC*) and the ELF Procedure in Chapter 4. Thus, the coefficient C_{sm} in Eq. 5-1 and the distribution equations, Eq. 5-4 and 5-4a, are the counterparts of Eq. 4-1, 4-6, and 4-6a. This correspondence helps clarify the fact that the simplified modal analysis contained in Chapter 5 is simply an attempt to specify the equivalent lateral forces on a building in a way that directly reflects the individual dynamic characteristics of the building. Once the story shears and other response variables for each of the important modes are determined and combined to produce design values, the design values are used in basically the same manner as the equivalent lateral forces given in Chapter 4.

The Modal Analysis Procedure specified in Chapter 5 is simplified from the general case by restricting consideration to lateral motion in a plane. As noted in Sec. 5.2, only one degree of freedom is required per floor for this type of motion. The effects of the horizontal component of ground motion perpendicular to the direction under consideration, the vertical component of ground motion, and the torsional motions of the building are all considered in the same simple manner as in the ELF Procedure.

5.3 MODES

This section defines the number of modes to be used in the analysis. For many structures, including low-rise buildings and structures of moderate height, three modes of vibration in each direction are nearly always sufficient to determine design values of the earthquake response of the building. For buildings of only one or two stories, a number of modes equal to the number of stories suffices for purposes of design and, hence, the last phrase. For high-rise structures, however, more than three modes may be required to adequately determine the forces for design. In this case, all modes having natural periods larger than 0.40 second are to be used. For very tall or very flexible structures, it may be necessary to consider six or more modes in each direction.

Although this section is intended to specify the minimum number of modes to be considered, there may be instances in which the designer should include additional modes in the analysis in order to obtain a more reliable indication of the possible earthquake response of the structure.

5.4 PERIODS

Natural periods of vibration are required for each of the modes used in the subsequent calculations. These are needed to determine the modal coefficients C_{sm} from Eq. 5-3. Because the periods of the modes contemplated in these provisions are those associated with moderately large, but still essentially linear, building response, the period calculations should include only those elements that are effective at these amplitudes. Such periods may be longer than 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 nonstructural and architectural components of the building at small amplitudes. During response to strong ground-shaking, however, measured responses of buildings have shown that the periods lengthen, indicating the loss of the stiffness contributed by those components.

There exists a wide variety of methods for calculation of natural periods and associated mode shapes, and no one particular method is required by the *Provisions*. It is essential, however, that the method used be one based on generally accepted principles of mechanics such as those given in well known textbooks on structural dynamics and vibrations (Clough and Penzien, 1975; Newmark and Rosenblueth, 1971; Thomson, 1965; Wiegel, 1970). Although it is expected that in many cases computer programs, whose accuracy and reliability are documented and widely recognized, will be used to calculate the required natural periods and associated mode shapes, their use is not required.

5.5 MODAL BASE SHEAR

A central feature of modal analysis is that the earthquake response is considered as a combination of the independent responses of the building vibrating in each of its important modes. As the building vibrates back and forth in a particular mode at the associated period, it experiences maximum values of base shear, interstory drifts, floor displacements, base (overturning) moments, etc. In this section, the base shear in the m^{th} mode is specified as the product of the modal seismic coefficient C_{sm} and the effective weight $\bar{W}_m$ for the mode. The coefficient C_{sm} is determined for each mode from Eq. 5-3 using the associated period of the mode, T_m, in addition to the factors A_v, S, and R, which are discussed elsewhere in the *Commentary*. An exception to this procedure occurs for higher modes of those buildings that have periods shorter than 0.3 second and that are founded on Type S_3 or S_4 soils. For such modes, Eq. 5-3a is used. Equation 5-3a gives values ranging from 0.8 A_a/R for very short periods to 2.0 A_a/R for T_m = 0.3. Comparing these values to the limiting values of C_s of 2.0 A_a/R for Type S_3 soils as specified following Eq. 5-3, it is seen that the use of Eq. 5-3a, when applicable, reduces the modal base shear. This is an approximation introduced in consideration of the conservatism embodied in using the spectral shape specified by Eq. 5-3 and its limiting values. The spectral shape so defined is a conservative approximation to average spectra that are known to first ascend, level off, and then decay as period increases. Equation 5-3 and its limiting values conservatively replace the ascending portion for small periods by a level portion. For Type S_1 and S_2 soils, the ascending portion of the spectra is completed by the time the period reaches a small value near 0.1 or 0.2 second. On the other hand, for soft soils the ascent may not be completed until a larger period is reached. Equation 5-3a is then a replacement for the spectral shape for Type S_3 and S_4 soils and short periods that is more consistent with spectra for measured accelerations. It was introduced because it was judged unnecessarily conservative to use Eq. 5-3 for modal analysis in the case of Type S_3 or S_4 soils. The effective modal gravity load given in Eq. 5-2 can be interpreted as specifying the portion of the weight of the building that participates in the vibration of each mode. It is noted that Eq. 5-2 gives values of $\bar{W}_m$ that are independent of how the modes are normalized.

The final equation of this section, Eq. 5-3b, is to be used if a modal period exceeds 4 seconds. It can be seen that Eq. 5-3b and 5-3 coincide at T_m = 4 seconds so that the effect of using Eq. 5-3b is to provide a more rapid decrease in C_{sm} as a function of the known characteristics of earthquake response spectra at intermediate and long periods. At intermediate periods, the average velocity spectrum of strong earthquake motions from large (magnitude 6.5 and larger) earthquakes is approximately constant, which implies that C_{sm} should decrease as $1/T_m$. Equation 5-3 decreases as $1/T_m^{2/3}$ for reasons discussed in Sec. 4.2 of the "Chapter 4 Commentary," and this slower rate of decrease, if extended to very long periods, would result in an unbalanced degree of conservatism in the modal force for very tall buildings. In addition, for very long periods, the average displacement spectrum of strong earthquake motions becomes constant which implies that C_{sm}, a form of acceleration spectrum, should decay as $1/T_m^2$. The period at which the displacement response spectrum becomes constant depends on the size of the earthquake, being larger for great earthquakes, and a representative period of 4 seconds was chosen to make the transition.

5.6 MODAL FORCES, DEFLECTIONS, AND DRIFTS

This section specifies the forces and displacements associated with each of the important modes of response.

Modal forces at each level are given by Eq. 5-4 and 5-4a and are expressed in terms of the gravity load assigned to the floor, the mode shape, and the modal base shear V_m. In applying the forces F_{xm} to the building, the direction of the forces is controlled by the algebraic sign of ϕ_{xm}. Hence, the modal forces for the fundamental mode will all act in the same direction, but modal forces for the second and higher modes will change direction as one moves up the building. The form of Eq. 5-4 is somewhat different from that usually employed in standard references and shows clearly the relation between the modal forces and the modal base shear. It therefore is a convenient form for calculation and highlights the similarity to Eq. 4-6a in the ELF Procedure.

The modal deflections at each level are specified by Eq. 5-5. These are the displacements caused by the modal forces F_{xm} considered as static forces and are representative of the maximum amplitudes of modal response for the essentially elastic motions envisioned within the concept of the seismic response modification coefficient R. This is also a logical point to calculate the modal drifts, which are required in Sec. 5.8. If the mode under consideration dominates the earthquake response, the modal deflection under the strongest motion contemplated by the *Provisions* can be estimated by multiplying by the deflection amplification factor C_d. It should be noted also that δ_{xm} is proportional to ϕ_{xm} (this can be shown with algebraic substitution for F_{xm} in Eq. 5-6) and will therefore change direction up and down the structure for the higher modes.

5.7 MODAL STORY SHEARS AND MOMENTS

This section merely specifies that the forces of Eq. 5-4 should be used to calculate the shears and moments for each mode under consideration. In essence, the forces from Eq. 5-4 are applied to each mass, and linear static methods are used to calculate story shears and story overturning moments. The base shear that results from the calculation should check with Eq. 5-1.

5.8 DESIGN VALUES

This section specifies the manner in which the values of story shear, moment, and drift quantities and the deflection at each level are to be combined. The method used, in which the design value is the square root of the sum of the squares of the modal quantities, was selected for its simplicity and its wide familiarity (Clough and Penzien, 1975; Newmark and Rosenblueth, 1971; Wiegel, 1970). In general, it gives satisfactory results, but it is not always a conservative predictor of the earthquake response inasmuch as more adverse combinations of modal quantities than are given by this method of combination can occur. The most common instance where combination by use of the square root of the sum of the squares is unconservative occurs when two modes have very nearly the same natural period. In this case, the responses are highly correlated and the designer should consider combining the modal quantities more conservatively (Newmark and Rosenblueth, 1971).

This section also limits the reduction of base shear that can be achieved by modal analysis compared to use of the ELF Procedure. Some reduction, where it occurs, is thought justified because the modal analysis gives a somewhat more accurate representation of the earthquake response. Some limit to any such possible reduction that may occur from the calculation of longer natural periods is necessary because the actual periods of vibration may not be as long, even at moderately large amplitudes of motion, due to the stiffening effects of elements not a part of the seismic resisting system and of nonstructural and architectural components. The limit is imposed by comparison to the ELF Procedure with a 20 percent increase in the factor C_a.

5.9 HORIZONTAL SHEAR DISTRIBUTION AND TORSION

This section requires that the design story shears calculated in Sec. 5.8 and the torsional moments prescribed in Sec. 4.4 be distributed to the vertical elements of the seismic resisting system as specified in Sec. 4.4 and as elaborated on in the corresponding section of the "Chapter 4 Commentary." This is consistent with the assumption of planar motion used in this simplified version of modal analysis and is intended to provide resistance against torsional response.

However, lateral and torsional motions may be strongly coupled if the building is irregular in its plan configuration (see Sec. 3.4) or if the building, although regular in plan and even with nearly coincident centers of mass and resistance, has its lower natural frequencies nearly equal. The designer should account for the effects of torsion in such buildings in a more accurate manner using methods of modal analysis capable of at least three degrees of freedom per floor (two translational and one torsional). (See Sec. 3.4 of the "Chapter 3 Commentary.")

5.10 FOUNDATION OVERTURNING

Because story moments are calculated mode by mode (properly recognizing that the direction of forces F_{xm} is controlled by the algebraic sign of ϕ_{xm}) and then combined to obtain the design values of story moments, there is no reason for reducing these design moments. This is in contrast with reductions permitted in overturning moments calculated from equivalent lateral forces in the analysis procedures of Chapter 4 (see Sec. 4.5 of the "Chapter 4 Commentary"). However, in the design of the foundation, the overturning moment calculated at the foundation-soil interface may be reduced by 10 percent for the reasons mentioned in Sec. 4.5 of the "Chapter 4 Commentary."

5.11 P-DELTA EFFECTS

Section 4.6 of the "Chapter 4 Commentary" applies to this section. In addition, to obtain the story drifts when using the modal analysis procedure of Chapter 5, the story drift for each mode should be independently determined in each story (Sec. 5.8). The story drift should not be determined from the differential combined lateral building deflections since this latter procedure will tend to mask the higher mode effects in longer period structures.

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Chapter 6 Commentary

SOIL-STRUCTURE INTERACTION

GENERAL

For the 1988 Edition of the *Provisions*, the detailed procedures for incorporating the effects of soil-structure interaction in the determination of design earthquake forces are included as the "Appendix to Chapter 6." Use of these procedures in the design of most buildings is considered to be unnecessary; therefore, it was decided that they were too specialized to be included in the *Provisions* proper.

6A.1 BACKGROUND AND SCOPE OF THE "APPENDIX TO CHAPTER 6"

Statement of the Problem

Fundamental to the design provisions presented in Chapters 4 and 5 is the assumption that the motion experienced by the base of a structure during an earthquake is the same as the free-field ground motion, a term that refers to the motion that would occur at the level of the foundation if no structure was present. Strictly speaking, this assumption is true only for structures supported on essentially rigid ground. For structures supported on soft soil, the foundation motion generally is different from the free-field motion and may include an important rocking component in addition to a lateral or translational component. The rocking component may be particularly significant for tall structures.

A flexibly supported structure also differs from a rigidly supported structure in that a substantial part of its vibrational energy may be dissipated into the supporting medium by radiation of waves and by hysteretic action in the soil. The importance of the latter factor increases with increasing intensity of ground-shaking. There is, of course, no counterpart of this effect of energy dissipation in a rigidly supported structure.

The effects of soil-structure interaction accounted for in the "Appendix to Chapter 6" represent the difference in the response of the structure computed by assuming the motion of the foundation to be the same as the free-field ground motion and considering the modified or actual motion of the foundation. This difference depends on the characteristics of the free-field ground motion as well as on the properties of the structure and the supporting medium.

The interaction effects provided for herein should not be confused with the so-called "site effects." The latter effects refer to the fact that the characteristics of the free-field ground motion induced by a dynamic event at a given site are functions of the properties and geological features of the subsurface soil and rock. The interaction effects, on the other hand, refer to the fact that the dynamic response of a structure built on that site depends, in addition, on the interrelationship of the structural characteristics and the properties of the local underlying soil deposits. The site effects are reflected in the values of the seismic design coefficients employed in Chapters 4 and 5 and are accounted for only implicitly in the "Appendix to Chapter 6."

Possible Approaches to the Problem

Two different approaches may be used to assess the effects of soil-structure interaction. The first involves modifying the stipulated free-field design ground motion and evaluating the response of the given structure to the modified motion of the foundation whereas the second involves modifying the dynamic properties of the structure and evaluating the response of the modified structure to the prescribed free-field ground motion (Veletsos, 1977). When properly implemented, both approaches lead to equivalent results. However, the second approach, involving the use of the free-field ground motion, is more convenient for design purposes and provides the basis of the provisions presented in the "Appendix to Chapter 6."

Characteristics of Interaction

The interaction effects in the approach used here are expressed by an increase in the fundamental natural period of the structure and a change (usually an increase) in its effective damping. The increase in period results from the flexibility of the foundation soil whereas the change in damping results mainly from the effects of energy dissipation in the soil due to radiation and material damping. These statements can be clarified by comparing the responses of rigidly and elastically supported systems subjected to a harmonic excitation of the base. Consider a linear structure of weight W, lateral stiffness k, and coefficient of viscous damping c (shown in Figure C6A-1) and assume that it is supported by a foundation of weight W_o at the surface of a homogeneous, elastic halfspace. The foundation mat is idealized as a rigid circular plate of negligible thickness bonded to the supporting medium, and the columns of the structure are considered to be weightless and axially inextensible. Both the foundation weight and the weight of the structure are assumed to be uniformly distributed over circular areas of radius r. The base excitation is specified by the free-field motion of the ground surface. This is taken as a horizontally directed, simple harmonic motion with a period T_o and an acceleration amplitude a_m.

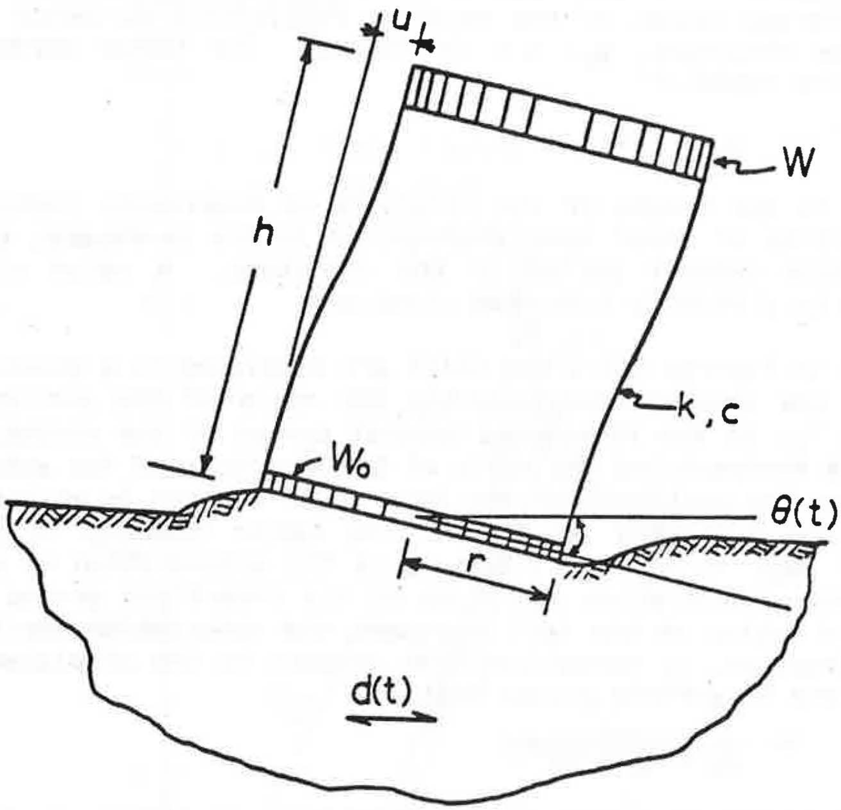

FIGURE 6A-1
Simple system investigated.

The configuration of this system, which has three degrees of freedom when flexibly supported and a single degree of freedom when fixed at the base, is specified by the lateral displacement and rotation of the foundation, y and θ, and by the displacement relative to the base of the top of the structure, u. The system may be viewed either as the direct model of a one-story building frame or, more generally, as a model of a multistory, multimode structure that responds as a single-degree-of-freedom system in its fixed-base condition. In the latter case, h must be interpreted as the distance from the base to the centroid of the inertia forces associated with the fundamental mode of vibration of the fixed-base structure and W, k, and c must be interpreted as its generalized or effective weight, stiffness, and damping coefficient, respectively. The relevant expressions for these quantities are given below.

The solid lines in Figures C6A-2 and C6A-3 represent response spectra for the steady-state amplitude of the total shear in the columns of the system considered in Figure C6A-1. Two different values of h/r and several different values of the relative flexibility parameter for the soil and the structure, ϕ_o, are considered. The latter parameter is defined by the equation:

$$\phi_o = h/v_s T, \qquad \text{(C6A-1)}$$

in which h is the height of the structure as previously indicated, v_s is the velocity of shear wave propagation in the halfspace, and T is the fixed-base natural period of the structure. A value of $\phi = 0$ corresponds to a rigidly supported structure.

The results in Figures C6A-2 and C6A-3 are displayed in a dimensionless form, with the abscissa representing the ratio of the period of the excitation, T_o, to the fixed-base natural period of the system, T, and the ordinate representing the ratio of the amplitude of the actual base shear, V, to the amplitude of the base shear induced in an infinitely stiff, rigidly supported structure. The latter quantity is given by the product ma_m, in which $m = W/g$, g is the acceleration of gravity, and a_m is the acceleration amplitude of the free-field ground motion. The inclined scales on the left represent the deformation amplitude of the superstructure, u, normalized with respect to the displacement amplitude of the free-field ground motion:

$$d_m = a_m T_o^2/4\pi^2. \qquad \text{(C6A-2)}$$

The damping of the structure in its fixed-base condition, β, is considered to be 2 percent of the critical value, and the additional parameters needed to characterize completely these solutions are identified in Veletsos and Meek (1974), from which these figures have been reproduced.

Comparison of the results presented in these figures reveals that the effects of soil-structure interaction are most strikingly reflected in a shift of the peak of the response spectrum to the right and a change in the magnitude of the peak. These changes, which are particularly prominent for taller structures and more flexible soils (increasing values of ϕ_o), can conveniently be expressed by an increase in the natural period of the system over its fixed-base value and by a change in its damping factor.

Also shown in these figures in dotted lines are response spectra for single-degree-of-freedom (SDF) oscillators, the natural period and damping of which have been adjusted so that the absolute maximum (resonant) value of the base shear and the associated period are in each case identical to those of the actual interacting systems. The base motion for the replacement oscillator is considered to be the same as the free-field ground motion. With the properties of the replacement SDF oscillator determined in this manner, it is important to note that the response spectra for the actual and the replacement systems are in excellent agreement over wide ranges of the exciting period on both sides of the resonant peak.

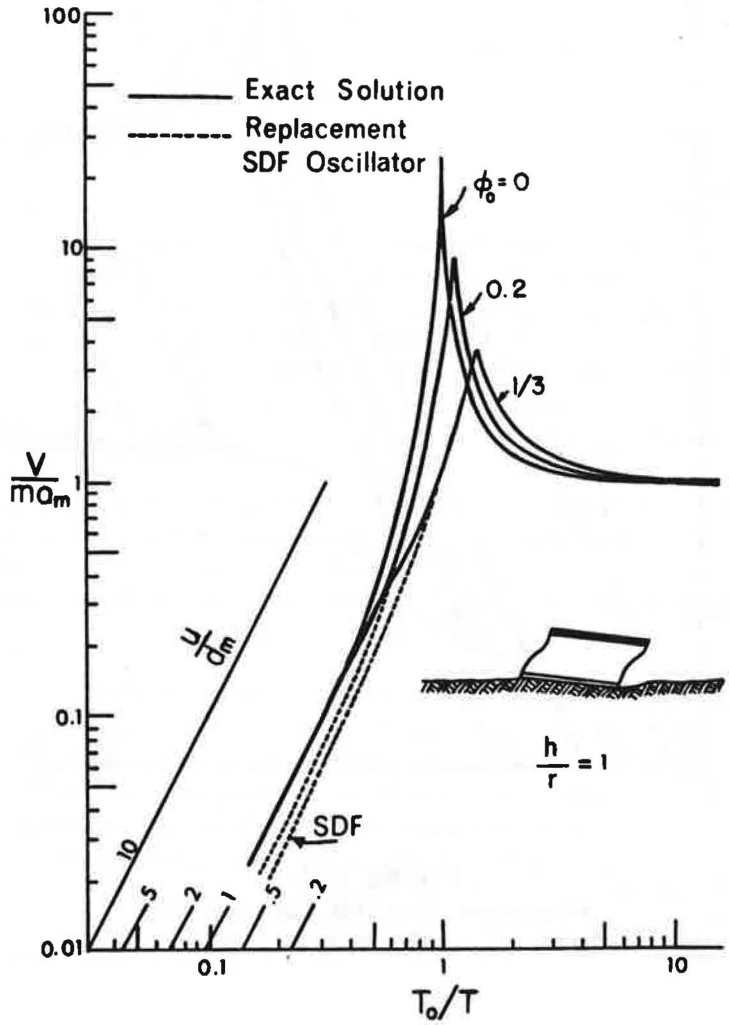

FIGURE C6A-2
Response spectra for systems with h/r = 1 (Veletsos and Meek, 1974).

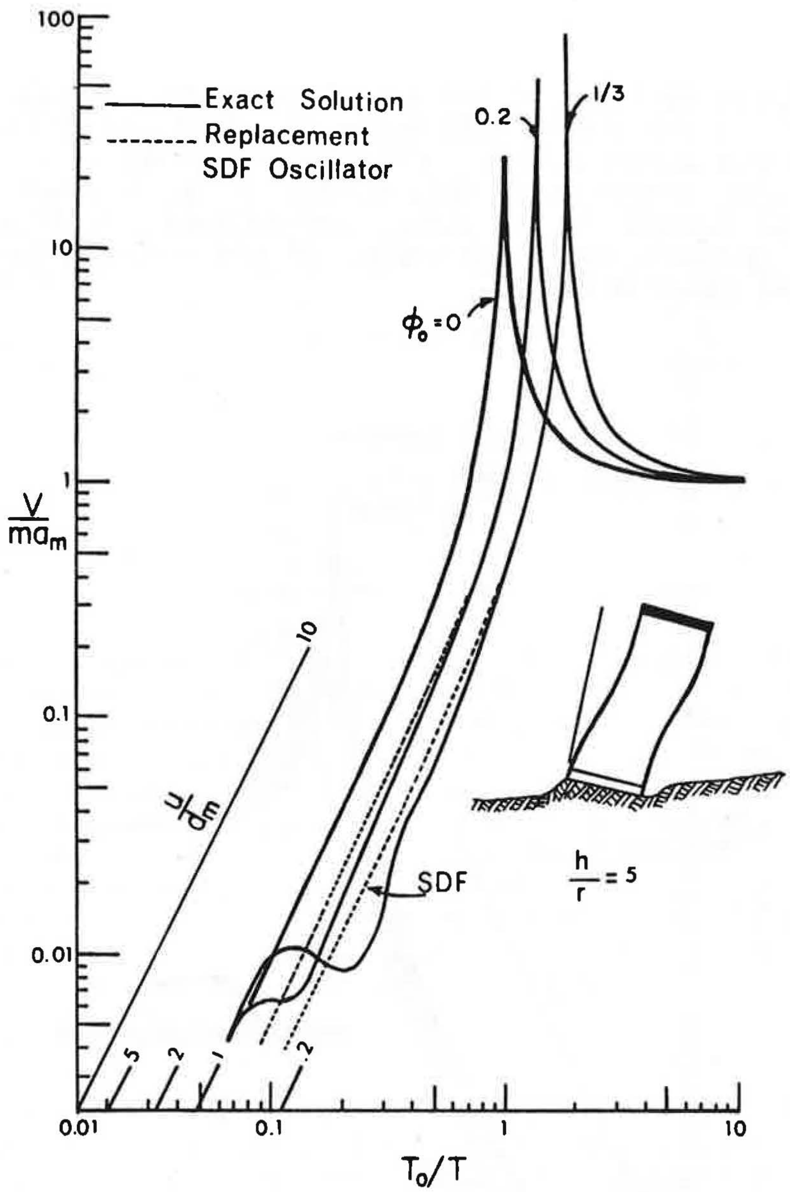

FIGURE C6A-3
Response spectra for systems with $h/r = 5$ (Veletsos and Meek, 1974).

In the context of Fourier analysis, an earthquake motion may be viewed as the result of superposition of harmonic motions of different periods and amplitudes. Inasmuch as the components of the excitation with periods close to the resonant period are likely to be the dominant contributors to the response, the maximum responses of the actual system and of the replacement oscillator can be expected to be in satisfactory agreement for earthquake ground motions as well. This expectation has been confirmed by the results of comprehensive comparative studies (Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975).

It follows that, to the degree of approximation involved in the representation of the actual system by the replacement SDF oscillator, the effects of interaction on maximum response may be expressed by an increase in the fundamental natural period of the fixed-base system and by a change in its damping value. In the following sections, the natural period of replacement oscillator is denoted by $\tilde{T}$ and the associated damping factor, by $\tilde{\beta}$. These quantities will also be referred to as the effective natural period and the effective damping factor of the interacting system. The relationships between $\tilde{T}$ and T and between $\tilde{\beta}$ and β are considered in Sec. 6A.2.1.1 and 6A.2.1.2.

Basis of Provisions and Assumptions

Current knowledge of the effects of soil-structure interactions is derived mainly from studies of systems of the type referred to above in which the foundation is idealized as a rigid mat. For foundations of this type, both surface-supported and embedded structures resting on uniform as well as layered soil deposits have been investigated (Bielak, 1975; Chopra and Gutierrez, 1974; Jennings and Bielak, 1973; Liu and Fagel, 1971; Parmelee et al., 1969; Roesset et al., 1973; Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975). However, only a small amount of information is available concerning the interaction effects for structures supported on spread footings or pile foundations (Blaney et al., n.d.; Novak, 1974; Rainer, 1975b). The provisions proposed in the "Appendix to Chapter 6" for the latter cases represent the best interpretation and judgment of the developers of the provisions regarding the current state of knowledge.

Fundamental to these provisions is the assumption that the structure and the underlying soil are bonded and remain so throughout the period of ground-shaking. It is further assumed that there is no soil instability or large foundation settlements. The design of the foundation in a manner to ensure satisfactory soil performance (e.g., to avoid soil instability and settlement associated with the compaction and liquefaction of loose granular soils), is beyond the scope of the "Appendix to Chapter 6." Finally, no account is taken of the interaction effects among neighboring structures.

Nature of Interaction Effects

Depending on the characteristics of the structure and the ground motion under consideration, soil-structure interaction may increase, decrease, or have no effect on the magnitudes of the maximum forces induced in the structure itself (Bielak, 1975; Jennings and Bielak, 1973; Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975). However, for the conditions stipulated in the development of the provisions for rigidly supported structures presented in Chapters 4 and 5, soil-structure interaction will <u>reduce</u> the design values of the base shear and moment from the levels applicable to a rigid-base condition. These forces therefore can be evaluated conservatively without the adjustments recommended in the "Appendix to Chapter 6."

Because of the influence of foundation rocking, however, the horizontal displacements relative to the base of the elastically supported structure may be larger than those of the corresponding fixed-base structure, and this may increase both the required spacing between buildings and the secondary design forces associated with the P-delta effects. Such increases generally are small.

Scope

Two procedures are used to incorporate effects of the soil-structure interaction. The first is an extension of the Equivalent Lateral Force Procedure presented in Chapter 4 and involves the use of equivalent lateral static forces. The second is an extension of the simplified Modal Analysis Procedure presented in Chapter 5. In the latter approach, the earthquake-induced effects are expressed as a linear combination of terms, the number of which is equal to the number of stories involved. Other more complex procedures also may be used, and these are outlined briefly at the end of this commentary on the "Appendix to Chapter 6." However, it is believed that the more involved procedures are justified only for unusual buildings of extreme importance and only when the results of the specified simpler approaches have revealed that the interaction effects are indeed of definite consequence in the design.

6A.2 EQUIVALENT LATERAL FORCE PROCEDURE

This procedure is similar to that used in the older SEAOC recommendations except that it incorporates several improvements (see the "Chapter 4 Commentary"). In effect, the procedure considers the response of the structure in its fundamental mode of vibration and accounts for the contributions of the higher modes implicitly through the choice of the effective weight of the structure and the vertical distribution of the lateral forces. The effects of soil-structure interaction are accounted for on the assumption that they influence only the contribution of the fundamental mode of vibration. For building structures, this assumption has been found to be adequate (Bielak, 1976; Jennings and Bielak, 1973; Veletsos, 1977).

6A.2.1 Base Shear

With the effects of soil-structure interaction neglected, the base shear is defined by Eq. 4-1 (Chapter 4):

$$V = C_s W, \tag{4-1}$$

in which W is the total dead weight of the building and of applicable portions of the design live load (as specified in Sec. 4.2) and C_s is the dimensionless seismic design coefficient (as defined by Eq. 4-2). The coefficient C_s depends on the seismic zone under consideration, the properties of the site, and the characteristics of the building itself. The latter characteristics include the fixed-base fundamental natural period of the structure, T; the associated damping factor, β; and the degree of permissible inelastic deformation. The damping factor does not appear explicitly in Eq. 4-2 because a constant value of β = 0.05 has been used for all structures for which the interaction effects are negligible. The degree of permissible inelastic action is reflected in the choice of the reduction factor, R.

It is convenient to rewrite Eq. 4-1 in the form:

$$V = C_s(T,\beta)\ \bar{W} + C_s(T,\beta)[W - \bar{W}], \tag{C6A-3}$$

where $\bar{W}$ represents the generalized or effective weight of the structure when vibrating in its fundamental natural mode. The terms in parentheses are used to emphasize the fact that C_s depends upon both T and β. The relationship between $\bar{W}$ and W is given below. The first term on the right side of Eq. C6A-3 approximates the contribution of the fundamental mode of vibration whereas the second term approximates the contributions of the higher natural modes.

Inasmuch as soil-structure interaction may be considered to affect only the contribution of the fundamental mode and inasmuch as this effect can be expressed by changes in the fundamental natural period and the associated damping of the system, the base shear for the interacting system, $\tilde{V}$, may be stated in a form analogous to Eq. C6A-3:

$$\tilde{V} = C_s(\tilde{T},\tilde{\beta})\ \bar{W} + C_s(T,\beta)[W - \bar{W}]. \tag{C6A-4}$$

The value of C_s in the first term of this equation should be evaluated for the natural period and damping of the elastically supported system, $\tilde{T}$ and $\tilde{\beta}$, respectively, and the value of C_s in the second term should be evaluated for the corresponding quantities of the rigidly supported system, T and β .

Before proceeding with the evaluation of the coefficients C_s in Eq. C6A-4, it is desirable to rewrite this formula in the same form as Eq. 6A-1. Making use of Eq. 4-1 and rearranging terms, the following expression for the reduction in the base shear is obtained:

$$\Delta V = [C_s(T,\beta) - C_s(\tilde{T},\tilde{\beta})]\ \bar{W}. \tag{C6A-5}$$

Within the ranges of natural period and damping that are of interest in studies of building response, the values of C_s corresponding to two different damping values but the same natural period (e.g., $\tilde{T}$), are related approximately as follows:

$$C_s\ (\tilde{T},\tilde{\beta}) = C_s(\tilde{T},\beta)\ (\beta/\tilde{\beta})^{0.4}. \qquad \text{(C6A-6)}$$

This expression, which appears to have been first proposed in Arias and Husid (1962), is in good agreement with the results of recent studies of earthquake response spectra for systems having different damping values (Newmark et al., 1973).

Substitution of Eq. C6A-6 in Eq. C6A-5 leads to:

$$\Delta V = [C_s\ (T,\beta) - C_s(\tilde{T},\beta)\ (\beta/\tilde{\beta})^{0.4}]\ \bar{W}, \qquad \text{(C6A-7)}$$

where both values of C_s are now for the damping factor of the rigidly supported system and may be evaluated from Eq. 4-2. If the values corresponding to the periods T and $\tilde{T}$ are denoted more simply as C_s and $\tilde{C}_s$, respectively, and if the damping factor β is taken as 0.05, Eq. C6A-7 reduces to Eq. 6A-2.

Note that $\tilde{C}_s$ in Eq. 6A-2 is smaller than or equal to C_s because Eq. 4-2 is a nonincreasing function of the natural period and $\tilde{T}$ is greater than or equal to T. Furthermore, since the minimum value of $\tilde{\beta}$ is taken as $\tilde{\beta} = \beta = 0.05$ (see statement following Eq. 6A-9), the shear reduction ΔV is a non-negative quantity. It follows that the design value of the base shear for the elastically supported structure cannot be greater than that for the associated rigid-base structure.

The effective weight of the building, $\bar{W}$, is defined by Eq. 5-2 (Chapter 5), in which ϕ_{im} should be interpreted as the displacement amplitude of the i^{th} floor when the structure is vibrating in its fixed-base fundamental natural mode. It should be clear that the ratio $\bar{W}/W$ depends on the detailed characteristics of the structure. A constant value of $\bar{W}$ = 0.7 W is recommended in the interest of simplicity and because it is a good approximation for typical buildings. As an example, it is noted that for a tall building for which the weight is uniformly distributed along the height and for which the fundamental natural mode increases linearly from the base to the top, the exact value of $\bar{W}$ = 0.75 W. Naturally, when the full weight of the structure is concentrated at a single level, $\bar{W}$ should be taken equal to W.

The maximum permissible reduction in base shear due to the effects of soil-structure interaction is set at 30 percent of the value calculated for a rigid-base condition. It is expected, however, that this limit will control only infrequently and that the calculated reduction, in most cases, will be less.

6A.2.1.1 Effective Building Period

Equation 6A-3 for the effective natural period of the elastically supported structure, $\tilde{T}$, is determined from analyses in which the superstructure is presumed to respond in its fixed-base fundamental mode and the foundation weight is considered to be negligible in comparison to the weight of the superstructure (Jennings and Bielak, 1973; Veletsos and Meek, 1974). The first term under the radical represents the period of the fixed-base structure. The first portion of the second term represents the contribution to $\tilde{T}$ of the translational flexibility of the foundation, and the last portion represents the contribution of the corresponding rocking flexibility. The quantities $\bar{k}$ and $\bar{h}$ represent, respectively, the effective stiffness and effective height of the structure, and K_y and K_θ represent the translational and rocking stiffnesses of the foundation.

Equation 6A-4 for the structural stiffness, $\bar{k}$, is deduced from the well known expression for the natural period of the fixed-base system:

$$T = 2\pi\sqrt{(1/g)(\bar{W}/\bar{k})}. \qquad \text{(C6A-8)}$$

The effective height, $\bar{h}$, is defined by Eq. 6A-13, in which ϕ_{i1} has the same meaning as the quantity ϕ_{im} in Eq. 5-2 (Chapter 5) when $m = 1$. In the interest of simplicity and consistency with the approximation used in the definition of $\bar{W}$, however, a constant value of $\bar{h} = 0.7\ h_n$ is recommended where h_n is the total height of the structure. This value represents a good approximation for typical buildings. As an example, it is noted that for tall buildings for which the fundamental natural mode increases linearly with height, the exact value of $\bar{h}$ is $2/3\ h_n$. Naturally, when the gravity load of the structure is effectively concentrated at a single level, h_n must be taken as equal to the distance from the base to the level of weight concentration.

Foundation stiffnesses depend on the geometry of the foundation-soil contact area, the properties of the soil beneath the foundation, and the characteristics of the foundation motion. Most of the available information on this subject is derived from analytical studies of the response of harmonically excited rigid circular foundations, and it is desirable to begin with a brief review of these results.

For circular mat foundations supported at the <u>surface</u> of a homogeneous halfspace, stiffnesses K_y and K_θ are given by:

$$K_y = (8\ \alpha_y)/(2 - \nu)Gr \qquad \text{(C6A-9)}$$

and

$$K_\theta = [8\ \alpha_\theta/3(1 - \nu)]Gr^3, \qquad \text{(C6A-10)}$$

where r is the radius of the foundation; G is the shear modulus of the halfspace; ν is its Poisson's ratio; and α_y and α_θ are dimensionless coefficients that depend on the period of the excitation, the dimensions of the foundation, and the properties of the supporting medium (Luco, 1974; Veletsos and Verbic, 1974; Veletsos and Wei, 1971). The shear modulus is related to the shear wave velocity, v_s, by the formula:

$$G = \gamma v_s^2/g, \tag{C6A-11}$$

in which γ is the unit weight of the material. The values of G, v_s, and ν should be interpreted as average values for the region of the soil that is affected by the forces acting on the foundation and should correspond to the conditions developed during the design earthquake. The evaluation of these quantities is considered further in subsequent sections. For statically loaded foundations, the stiffness coefficients α_y and α_θ are unity, and Eq. C6A-9 and C6A-10 reduce to:

$$K_y = 8\ Gr/(2 - \nu) \tag{C6A-12}$$

and

$$K_\theta = 8\ Gr^3/3(1 - \nu). \tag{C6A-13}$$

Studies of the interaction effects in structure-soil systems have shown that, within the ranges of parameters of interest for building structures subjected to earthquakes, the results are insensitive to the period-dependency of α_y and α_θ and that it is sufficiently accurate for practical purposes to use the static stiffnesses, defined by Eq. C6A-12 and C6A-13.

Foundation embedment has the effect of increasing the stiffnesses K_y and K_θ. For embedded foundations for which there is positive contact between the side walls and the surrounding soil, K_y and K_θ may be determined from the following approximate formulas:

$$K_y \approx [8\ Gr/(2 - \nu)][1 + (2/3)(d/r)] \tag{C6A-14}$$

and

$$K_\theta \approx [8\ Gr^3/3(- \nu)][1 + 2(d/r)], \tag{C6A-15}$$

in which d is the depth of embedment. These formulas are based on finite element solutions (Blaney et al., n.d.).

Both analyses and available test data (Erden, 1974) indicate that the effects of foundation embedment are sensitive to the condition of the backfill and that judgment must be exercised in using Eq. C6A-14 and C6A-15. For example, if a structure is embedded in such a way that there is no positive contact between the soil and the walls of the structure, or when any existing contact cannot reasonably be expected to remain effective during the stipulated design ground motion, stiffnesses K_y and K_θ should be determined from the formulas for surface-supported foundations. More generally, the quantity d in Eq. C6A-14 and C6A-15 should be interpreted as the effective depth of foundation embedment for the conditions that would prevail during the design earthquake.

The formulas for K_y and K_θ presented above are strictly valid only for foundations supported on reasonably uniform soil deposits. When the foundation rests on a stratum of soft soil underlain by a much stiffer, rock-like deposit with an abrupt increase in stiffness, K_y and K_θ may be determined from the two generalized formulas in which G is the shear modulus of the soft soil and D_s is the total depth of the stratum.

First, using Eq. C6A-16, K_y =

$$[8\ Gr/(2 - \nu)][1 + (2/3)(d/r)][1 + (1/2)(r/D_s)][1 + (5/4)(d/D_s)].$$

Second, using Eq. C6A-17, K_θ =

$$[8\ Gr^3/3(1 - \nu)][1 + 2(d/r)][1 + (1/6)(r/D_s)][1 + 0.7(d/D_s)].$$

These formulas are based on analyses of a stratum supported on a rigid base (Elsabee et al.,1977; Kausel and Roesset, 1975).

The information for circular foundations presented above may be applied to mat foundations of arbitrary shapes provided the following changes are made:

1. The radius r in the expressions for K_y in Eq. 6A-7 is replaced by the quantity:

   $$r_a = \sqrt{A_o/\pi},$$

   which represents the radius of a disk that has the area, A_o, of the actual foundation.

2. The radius r in the expressions for K_θ in Eq. 6A-8 is replaced by the quantity:

   $$r_m = \sqrt[4]{I_o/\pi},$$

   which represents the radius of a disk that has the moment of inertia, I_o, of the actual foundation.

For footing foundations, stiffnesses K_y and K_θ are computed by summing the contributions of the individual footings. If it is assumed that the foundation behaves as a rigid body and that the individual footings are widely spaced so that they act as independent units, the following formulas are obtained:

$$K_y = \Sigma k_{yi} \qquad \text{(C6A-18)}$$

and

$$K_\theta = \Sigma k_{xi} y_i^2 + \Sigma k_{\theta i}. \qquad \text{(C6A-19)}$$

The quantity k_{yi} represents the horizontal stiffness of the i^{th} footing; k_{xi} and $k_{\theta i}$ represent, respectively, the corresponding vertical and rocking stiffnesses; and y_i represents the normal distance from the centroid of the i^{th} footing to the rocking axis of the foundation. The summations are considered to extend over all footings. The contribution to K_θ of the rocking stiffnesses of the individual footings, $k_{\theta i}$, generally is small and may be neglected.

The stiffnesses k_{yi}, k_{xi}, and $k_{\theta i}$ are defined by the formulas:

$$k_{yi} = [8G_i r_{ai}/(2 - \nu)][1 + 2/3 d_i/r_{ai}], \qquad \text{(C6A-20)}$$

$$k_{xi} = [4G_i r_{ai}/(1 - \nu)][1 + 0.4 d_i/r_a], \qquad \text{(C6A-21)}$$

and

$$k_{\theta i} = [8G_i r_{mi}^3/2(1 - \nu)][1 + 2d_i/r_{mi}], \qquad \text{(C6A-22)}$$

in which d_i is the depth of effective embedment for the i^{th} footing; G_i is the shear modulus of the soil beneath the i^{th} footing; $r_{ai} = \sqrt{A_{oi}/\pi}$ is the radius of a circular footing that has the area of the i^{th} footing, A_{oi}; and r_{mi} equals $\sqrt[4]{4I_{oi}/\pi}$ = the radius of a circular footing, the moment of inertia of which about a horizontal centroidal axis is equal to that of the i^{th} footing, I_{oi}, in the direction in which the response is being evaluated.

For surface-supported footings and for embedded footings for which the side wall contact with the soil cannot be considered to be effective during the stipulated design ground motion, d_i in these formulas should be taken as zero. Furthermore, the values of G_i should be consistent with the stress levels expected under the footings and should be evaluated with due regard for the effects of the dead loads involved. This matter is considered further in subsequent sections.

For closely spaced footings, consideration of the coupling effects among footings will reduce the computed value of the overall foundation stiffness. This reduction will, in turn, increase the fundamental natural period of the system, $\tilde{T}$, and decrease the value of ΔV, the amount by which the base shear is reduced due to soil-structure interaction. It follows that the use of Eq. C6A-18 and C6A-19 will err on the conservative side in this case. The degree of conservatism involved, however, will partly be compensated by the presence of a basement slab that, even when it is not tied to the structural frame, will increase the overall stiffness of the foundation.

The values of K_y and K_θ for pile foundations can be computed in a manner analogous to that described in the preceding section by evaluating the horizontal, vertical, and rocking stiffnesses of the individual piles, k_{yi}, k_{xi} and $k_{\theta i}$, and by combining these stiffnesses in accordance with Eq. C6A-18 and C6A-19.

The individual pile stiffnesses may be determined from field tests or analytically by treating each pile as a beam on an elastic subgrade. Numerous formulas are available in the literature (Nair et al., 1969) that express these stiffnesses in terms of the modulus of the subgrade reaction and the properties of the pile itself. Although they differ in appearance, these formulas lead to practically similar results. These stiffnesses are typically expressed in terms of the stiffness of an equivalent freestanding cantilever, the physical properties and cross-sectional dimensions of which are the same as those of the actual pile but the length of which is adjusted appropriately. The effective lengths of the equivalent cantilevers for horizontal motion and for rocking or bending motion are slightly different but are often assumed to be equal. On the other hand, the effective length in vertical motion is generally considerably greater. For further details, the reader is referred to Nair et al. (1969).

The soil properties of interest are the shear modulus, G, or the associated shear wave velocity, v_s; the unit weight, γ; and Poisson's ratio, ν. These quantities are likely to vary from point to point of a construction site, and it is necessary to use average values for the soil region that is affected by the forces acting on the foundation. The depth of significant influence is a function of the dimensions of the foundation base and of the direction of the motion involved. The effective depth may be considered to extend to about $4r_a$ below the foundation base for horizontal and vertical motions and to about $1.5r_m$ for rocking motion. For mat foundations, the effective depth is related to the total plan dimensions of the mat whereas for buildings supported on widely spaced spread footings, it is related to the dimensions of the individual footings. For closely spaced footings, the effective depth may be determined by superposition of the "pressure bulbs" induced by the forces acting on the individual footings.

Since the stress-strain relations for soils are nonlinear, the values of G and v_s also are functions of the strain levels involved. In the formulas presented above, G should be interpreted as the secant shear modulus corresponding to the significant strain level in the affected region of the foundation soil. The approximate relationship of this modulus to the modulus G_o corresponding to small amplitude strains (of the order of 10^{-3} percent or less) is given in Table 6A-1. The backgrounds of this relationship and of the corresponding relationship for v_s/v_{so} are identified below.

The low amplitude value of the shear modulus, G_o, can most conveniently be determined from the associated value of the shear wave velocity, v_{so}, by use of Eq. C6A-11. The latter value may be determined approximately from empirical relations or more accurately by means of field tests or laboratory tests.

The quantities G_o and v_{so} depend on a large number of factors (Hardin and Black, 1968; Hardin and Drnevich, 1972; Richart et al., n.d.), the most important of which are the void ratio, e, and the average confining pressure, $\bar{\sigma}_o$. The value of the latter pressure at a given depth beneath a particular building foundation may be expressed as the sum of two terms as follows:

$$\bar{\sigma}_o = \bar{\sigma}_{os} + \bar{\sigma}_{ob}, \tag{C6A-23}$$

in which $\bar{\sigma}_{os}$ represents the contribution of the weight of the soil and σ_{ob} represents the contribution of the superimposed weight of the building and foundation. The first term is defined by the formula:

$$\bar{\sigma}_{os} = (1 + 2\ K_o/3)\gamma' x, \tag{C6A-24}$$

in which x is the depth of the soil below the ground surface, γ' is the average effective unit weight of the soil to the depth under consideration, and K_o is the coefficient of horizontal earth pressure at rest. For sands and gravel, K_o has a value of 0.5 to 0.6 whereas for soft clays, $K_o \approx 1.0$. The pressures $\bar{\sigma}_{ob}$ developed by the weight of the building can be estimated from the theory of elasticity (Poulos and Davis, 1974). In contrast to $\bar{\sigma}_{os}$ which increases linearly with depth, the pressures $\bar{\sigma}_{ob}$ decrease with depth. As already noted, the value of v_{so} should correspond to the average value of $\bar{\sigma}_o$ in the region of the soil that is affected by the forces acting on the foundation.

For clean sands and gravels having e < 0.80, the low-amplitude shear wave velocity can be calculated approximately from the formula:

$$v_{so} = c_1\ (2.17 - e)(\bar{\sigma})^{0.25}, \tag{C6A-25}$$

in which c_1 equals 78.2 when $\bar{\sigma}$ is in lb/ft^2 and v_{so} is in ft/sec; c_1 equals 160.4 when $\bar{\sigma}$ is in kg/cm^2 and v_{so} is in m/sec; and c_1 equals 51.0 when $\bar{\sigma}$ is in kN/m^2 and v_{so} is in m/sec.

For angular-grained cohesionless soils (e > 0.6), the following empirical equation may be used:

$$v_{so} = c_2\ (2.97 - e)(\bar{\sigma})^{0.25}, \tag{C6A-26}$$

in which c_2 equals 53.2 when $\bar{\sigma}$ is in lb/ft^2 and v_{so} is in ft/sec; c_2 equals 109.7 when $\bar{\sigma}$ is in kg/cm^2 and v_{so} is in m/sec; and c_2 equals 34.9 when $\bar{\sigma}$ is in kN/m^2 and v_{so} is in m/sec.

Equation C6A-26 also may be used to obtain a first-order estimate of v_{so} for normally consolidated cohesive soils. A crude estimate of the shear modulus, G_o, for such soils may also be obtained from the relationship:

$$G_o = 1{,}000\ S_u, \tag{C6A-27}$$

in which S_u is the shearing strength of the soil as developed in an unconfined compression test. The coefficient 1,000 represents a typical value, which varied from 250 to about 2,500 for tests on different soils (Hara et al., 1974; Hardin and Drnevich, 1972).

These empirical relations may be used to obtain preliminary, order-of-magnitude estimates. For more accurate evaluations, field and/or laboratory determinations may be required.

Field evaluations of the variations of v_{so} throughout the construction site can be carried out by standard seismic refraction methods or by the cross-hole method. The cross-hole method (Ballard and McLean, 1975; Stokoe and Woods, 1972) provides information from undisturbed soils below the proposed location of a particular building foundation. The method permits evaluation of v_{so} in layered soils and is not affected by the presence of water in the soil. The low-amplitude procedure is relatively inexpensive and easy to use. The disadvantage of this method is that v_{so} is determined only for the stress conditions existing at the time of the test (usually $\bar{\sigma}_{so}$). The effect of the changes in the stress conditions caused by construction must be considered by use of Eq. C6A-23 and Eq. C6A-25 or C6A-26 to adjust the field measurement of v_{so} to correspond to the prototype situations. The influence of large-amplitude shearing strains may be evaluated from laboratory tests or approximated through the use of Table 6A-1. This matter is considered further in the next two sections.

Laboratory tests to evaluate v_{so} are usually carried out with resonant column devices (Richart et al., n.d.). Such tests may be used to assess the effects of changes in confining pressures, shearing strain amplitudes, stress histories, temperature, and other variables. Consequently, they can easily simulate variations in prototype loading conditions. They are particularly useful in establishing the effects of changes in confining pressures. In fact, Eq. C6A-25 and C6A-26 were developed from the results of such tests.

An increase in the shearing strain amplitude is associated with a reduction in the secant shear modulus, G, and the corresponding value of v_s. Extensive laboratory tests (see, for example, Anderson and Richart, 1976; Hardin and Drnevich, 1972; Kuribayashi et al., 1974) have established the magnitudes of the reductions in v_s for both sands and clays as the shearing strain amplitude increases.

The results of such tests form the basis for the information presented in Table 6A-1. For each severity of anticipated ground-shaking, represented by the effective peak acceleration coefficients A_a and A_v, a representative value of shearing strain amplitude was developed. A conservative value of v_s/v_{so} that is appropriate to that strain amplitude then was established. It should be emphasized that the values in Table 6A-1 are first order approximations. More precise evaluations would require laboratory tests on undisturbed samples from the site and studies of wave propagation for the site to determine the magnitude of the soil strains induced.

It is satisfactory to assume Poisson's ratio for soils as: $\nu = 0.33$ for clean sands and gravels, $\nu = 0.40$ for stiff clays and cohesive soils, and $\nu = 0.45$ for soft clays. The use of an average value of $\nu \approx 0.4$ also will be adequate for practical purposes.

Regarding an alternative approach, note that Eq. 6A-5 for the period $\tilde{T}$ of buildings supported on mat foundations was deduced from Eq. 6A-3 by making use of Eq. C6A-12 and C6A-13, with Poisson's ratio taken as $\nu = 0.4$ and with the radius r interpreted as r_a in Eq. C6A-12 and as r_m in Eq. C6A-13. For a nearly square foundation, for which $r_a \approx r_m \approx r$, Eq. 6A-5 reduces to:

$$\tilde{T} = T\sqrt{[1 + 25\,\alpha\,(r\bar{h}/v_s^2T^2)][1 + (1.12\,\bar{h}/r)^2]}. \qquad \text{(C6A-28)}$$

The value of the relative weight parameter, α, is likely to be in the neighborhood of 0.15 for typical buildings.

6A.2.1.2 Effective Damping

Equation 6A-9 for the overall damping factor of the elastically supported structure, $\tilde{\beta}$, was determined from analyses of the harmonic response at resonance of simple systems of the type considered in Figures C6A-2 and C6A-3. The result is an expression of the form (Bielak, 1975; Veletsos and Nair, 1975):

$$\tilde{\beta} = \beta_o + 8/(\tilde{T}/T)^3, \qquad \text{(C6A-29)}$$

in which β_o represents the contribution of the foundation damping, considered in greater detail in the following paragraphs, and the second term represents the contribution of the structural damping. The latter damping is assumed to be of the viscous type. Equation 6A-9 corresponds to the value of $\beta = 0.05$ used in the development of the response spectra for rigidly supported systems employed in Chapter 4.

The foundation damping factor, β_o, incorporates the effects of energy dissipation in the soil due to the following sources: the radiation of waves away from the foundation, known as radiation or geometric damping, and the hysteretic or inelastic action in the soil, also known as soil material damping. This factor depends on the geometry of the foundation-soil contact area and on the properties of the structure and the underlying soil deposits.

For mat foundations of circular plan that are supported at the surface of reasonably uniform soils deposits, the three most important parameters which affect the value of β_o are: the ratio $\tilde{T}/T$ of the fundamental natural periods of the elastically supported and the fixed-base structures, the ratio $\bar{h}/r$ of the effective height of the structure to the radius of the foundation, and the damping capacity of the soil. The latter capacity is measured by the dimensionless ratio $\Delta W_s/W_s$, in which ΔW_s is the area of the hysteresis loop in the stress-strain diagram for a soil specimen undergoing harmonic shearing deformation and W_s is the strain energy stored in a linearly elastic material subjected to the same maximum stress and strain (i.e., the area of the triangle in the stress-strain diagram between the origin and the point of the maximum induced stress and strain). This ratio is a function of the magnitude of the imposed peak strain, increasing with increasing intensity of excitation or level of strain.

The variation of β_o with $\tilde{T}/T$ and $\bar{h}/r$ is given in Figure 6A-1 for two levels of excitation. The dashed lines, which are recommended for values of the effective ground acceleration coefficient, A_v, equal to or less than 0.10, correspond to a value of $\Delta W_s/W_s \approx 0.3$, whereas the solid lines, which are recommended for A_v values equal to or greater than 0.20, correspond to a value of $\Delta W_s/W_s \approx 1$. These curves are based on the results of extensive parametric studies (Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975) and represent average values. For the ranges of parameters that are of interest in practice, however, the dispersion of the results is small.

For mat foundations of arbitrary shape, the quantity r in Figure 6A-1 should be interpreted as a characteristic length that is related to the length of the foundation, L_o, in the direction in which the structure is being analyzed. For short, squatty structures for which $\bar{h}/L_o \leq 0.5$, the overall damping of the structure-foundation system is dominated by the translational action of the foundation, and it is reasonable to interpret r as r_a, the radius of a disk that has the same area as that of the actual foundation (see Eq. 6A-7). On the other hand, for structures with $\bar{h}/L_o \geq 1$, the interaction effects are dominated by the rocking motion of the foundation, and it is reasonable to define r as the radius r_m of a disk whose static moment of inertia about a horizontal centroidal axis is the same as that of the actual foundation normal to the direction in which the structure is being analyzed (see Eq. 6A-8).

Subject to the qualifications noted in the following section, the curves in Figure 6A-1 also may be used for embedded mat foundations and for foundations involving spread footings or piles. In the latter cases, the quantities A_o and I_o in the expressions for the characteristic foundation length, r, should be interpreted as the area and the moment of inertia of the load-carrying foundation.

In the evaluation of the overall damping of the structure-foundation system, no distinction has been made between surface-supported foundations and embedded foundations. Since the effect of embedment is to increase the damping capacity of the foundation (Bielak, 1975; Novak, 1974; Novak and Beredugo, 1972) and since such an increase is associated with a reduction in the magnitude of the forces induced in the structure, the use of the recommended provisions for embedded structures will err on the conservative side.

Sec. 6A.2.1.2

There is one additional source of conservatism in the application of the recommended provisions to buildings with embedded foundations. It results from the assumption that the free-field ground motion at the foundation level is independent of the depth of foundation embedment. Actually, there is evidence to the effect that the severity of the free-field excitation decreases with depth (Seed et al., 1977). This reduction is ignored both in the "Appendix to Chapter 6" and in the provisions for rigidly supported structures presented in Chapters 4 and 5.

Equations 6A-9 and C6A-29, in combination with the information presented in Figure 6A-1, may lead to damping factors for the structure--soil system, $\tilde{\beta}$, that are smaller than the structural damping factor, β. However, since the representative value of $\beta = 0.05$ used in the development of the design provisions for rigidly supported structures is based on the results of tests on actual buildings, it reflects the damping of the full structure-soil system, not merely of the component contributed by the superstructure. Thus, the value of $\tilde{\beta}$ determined from Eq. 6A-9 should never be taken less than β, and a low bound of $\tilde{\beta} = \beta = 0.05$ has been imposed. The use of values of $\tilde{\beta} > \beta$ is justified by the fact that the experimental values correspond to extremely small-amplitude motions and do not reflect the effects of the higher soil damping capacities corresponding to the large soil strain levels associated with the design ground motions. The effects of the higher soil damping capacities are appropriately reflected in the values of β_o presented in Figure 6A-1.

There are, however, some exceptions. For foundations involving a soft soil stratum of reasonably uniform properties underlain by a much stiffer, rock-like material with an abrupt increase in stiffness, the radiation damping effects are practically negligible when the natural period of vibration of the stratum in shear,

$$T_s = 4D_s/v_s, \tag{C6A-30}$$

is smaller than the natural period of the flexibly supported structure, $\tilde{T}$. The quantity D_s in this formula represents the depth of the stratum. It follows that the values of β_o presented in Figure 6A-1 are applicable only when:

$$T_s/\tilde{T} = 4D_s/v_s\tilde{T} \gg 1. \tag{C6A-31}$$

For

$$T_s/\tilde{T} = 4D_s/v_s\tilde{T} < 1, \tag{C6A-32}$$

the effective value of the foundation damping factor, β_o', is less than β_o, and it is approximated by the second degree parabola defined by Eq. 6A-10.

For $T_s/\tilde{T} = 1$, Eq. 6A-10 leads to $\beta_o' = \beta_o$ whereas for $T_s/\tilde{T} = 0$, it leads to $\beta_o' = 0$, a value that clearly does not provide for the effects of material soil damping. It may be expected, therefore, that the computed values of β_o' corresponding to small values of $T_s/\tilde{T}$ will be conservative. The conservatism involved, however, is partly compensated by the requirement that $\tilde{\beta}$ be no less than $\tilde{\beta} = \beta = 0.05$.

6A.2.2-6A.2.3 Vertical Distribution of Seismic Forces and Other Effects

The vertical distributions of the equivalent lateral forces for flexibly and rigidly supported structures are generally different. However, the differences are inconsequential for practical purposes, and it is recommended that the same distribution be used in both cases, changing only the magnitude of the forces to correspond to the appropriate base shear. A greater degree of refinement in this step would be inconsistent with the approximations embodied in the provisions for rigidly supported structures.

With the vertical distribution of the lateral forces established, the overturning moments and the torsional effects about a vertical axis are computed as for rigidly supported structures.

Equation 6A-11 for the lateral floor displacements relative to the base is similar to that specified for rigidly supported structures except that it includes the contribution of the foundation rotation θ_o. This rotation is defined by the equation:

$$\theta_o = \tilde{M}_o/K_\theta = (\tilde{V}/V)\ (M_o/K_\theta), \qquad \text{(C6A-33)}$$

in which $\tilde{M}_o$ is the overturning moment at the base of the fixed-base structure computed from the modified or reduced seismic forces and M_o is the corresponding moment computed from the unmodified forces. The latter moment should not include the reduction permitted in the design of the foundation. The quantity δ_x in Eq. 6A-11 represents the deflection at level h_x computed in accordance with the provisions of Chapter 4 using the unmodified seismic forces.

Story drifts and P-delta effects should be evaluated as for structures without interaction using the displacements that include the contribution of the foundation rotation.

6A.3 MODAL ANALYSIS PROCEDURE

Studies of the dynamic response of elastically supported multi-degree-of-freedom systems (Bielak, 1976; Chopra and Gutierrez, 1974; Veletsos, 1977) reveal that, within the ranges of parameters that are of interest in the design of building structures subjected to earthquakes, soil-structure interaction affects substantially only the response component contributed by the fundamental mode of vibration of the superstructure. In this section, the interaction effects are considered only in evaluating the contribution of the fundamental structural mode. The contributions of the higher modes are computed as if the structure were fixed at the base, and the maximum value of a response quantity is determined, as for rigidly supported structures, by taking the square root of the sum of the squares of the maximum modal contributions.

The interaction effects associated with the response in the fundamental structural mode are determined in a manner analogous to that used in the analysis of the equivalent lateral force method, except that the effective weight and effective height of the structure are computed so as to correspond exactly to those of the fundamental natural mode of the fixed-base structure. More specifically, $\bar{W}$ is computed from:

$$\bar{W} = \bar{W}_1 = (\Sigma\, w_i \phi_{i1})^2 / \Sigma\, w_i \phi_{i1}^2, \qquad \text{(C6A-34)}$$

which is the same as Eq. 5-2, and $\bar{h}$ is computed from Eq. 6A-13. The quantity ϕ_{i1} in these formulas represents the displacement amplitude of the i^{th} floor level when the structure is vibrating in its fixed-base fundamental natural mode. The structural stiffness, $\bar{k}$, is obtained from Eq. 6A-4 by taking $\bar{W} = \bar{W}_1$ and using for T the fundamental natural period of the fixed-base structure, T_1. The fundamental natural period of the interacting system, $\tilde{T}_1$, is then computed from Eq. 6A-3 (or Eq. 6A-5 when applicable) by taking $T = T_1$. The effective damping in the first mode, β, is determined from Eq. 6A-9 (and Eq. 6A-10 when applicable) in combination with the information given in Figure 6A-1. The quantity $\bar{h}$ in the latter figure is computed from Eq. 6A-13.

With the values of $\tilde{T}_1$ and $\tilde{\beta}_1$ established, the reduction in the base shear for the first mode, ΔV_1, is computed from Eq. 6A-2. The quantities C_s and $\tilde{C}_s$ in this formula should be interpreted as the seismic coefficients corresponding to the periods T_1 and $\tilde{T}_1$, respectively; $\tilde{\beta}$ should be taken equal to $\tilde{\beta}_1$; and $\bar{W}$ should be determined from Eq. C6A-34.

The sections on lateral forces, shears, overturning moments, and displacements follow directly from what has already been noted in this and the preceding sections and need no elaboration. It may only be pointed out that the first term within the brackets on the right side of Eq. 6A-14 represents the contribution of the foundation rotation.

6A.3.3 Design Values

The design values of the modified shears, moments, deflections, and story drifts should be determined as for structures without interaction by taking the square root of the sum of the squares of the respective modal contributions. In the design of the foundation, the overturning moment at the foundation-soil interface determined in this manner may be reduced by 10 percent as for structures without interaction.

The effects of torsion about a vertical axis should be evaluated in accordance with the provisions of Sec. 4.4 and the P-delta effects should be evaluated in accordance with the provisions of Sec. 4.6.2, using the story shears and drifts determined in Sec. 6A.3.2.

OTHER METHODS OF CONSIDERING THE EFFECTS OF SOIL-STRUCTURE INTERACTION

The procedures proposed in the preceding sections for incorporating the effects of soil-structure interaction provide sufficient flexibility and accuracy for practical applications. Only for unusual structures of major importance, and only when the provisions indicate that the interaction effects are of definite consequence in design, would the use of more elaborate procedures be justified.

Some of the possible refinements, listed in order of more or less increasing complexity, are:

1. Improve the estimates of the static stiffnesses of the foundation, K_y and K_θ, and of the foundation damping factor, β_0, by considering in a more precise manner the foundation type involved, the effects of foundation embedment, variations of soil properties with depth, and hysteretic action in the soil. Solutions may be obtained in some cases with analytical or semi-analytical formulations and in others by application of finite difference or finite element techniques (Blaney et al., 1974; Luco, 1974; Novak, 1974; Veletsos and Verbic, 1973). It should be noted, however, that these solutions involve approximations of their own that may offset, at least in part, the apparent increase in accuracy.

2. Improve the estimates of the average properties of the foundation soils for the stipulated design ground motion. This would require both laboratory tests on undisturbed samples from the site and studies of wave propagation for the site. The laboratory tests are needed to establish the actual variations with shearing strain amplitude of the shear modulus and damping capacity of the soil, whereas the wave propagation studies are needed to establish realistic values for the predominant soil strains induced by the design ground motion.

3. Incorporate the effects of interaction for the higher modes of vibration of the structure, either approximately by application of the procedures recommended in Bielak (1976), Roesset et al. (1973), and Tsai (1974) or by more precise analyses of the structure-soil system. The latter analyses may be implemented either in the time domain by application of the impulse

response functions presented in Veletsos and Verbic (1974). However, the frequency domain analysis is limited to systems that respond within the elastic range while the approach involving the use of the impulse response functions is limited, at present, to soil deposits that can adequately be represented as a uniform elastic halfspace. The effects of yielding in the structure and/or supporting medium can be considered only approximately in this approach by representing the supporting medium by a series of springs and dashpots whose properties are independent of the frequency of the motion and by integrating numerically the governing equations of motion (Parmelee et al., 1969).

4. Analyze the structure-soil system by finite element method (Seed et al., 1974 and 1977; Vaish and Chopra, 1974), taking due account of the nonlinear effects in both the structure and the supporting medium.

It should be emphasized that, while these more elaborate procedures may be appropriate in special cases for design verification, they involve their own approximations and do not eliminate the uncertainties that are inherent in the modeling of the structure-foundation-soil system and in the specification of the design ground motion and of the properties of the structure and soil.

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Chapter 7 Commentary

FOUNDATION DESIGN REQUIREMENTS

7.1 GENERAL

The minimum foundation design requirements that might be suitable when any consideration must be given to earthquake resistance are set forth in Chapter 7. It is difficult to separate foundation requirements for minimal earthquake resistance from the requirements for resisting normal vertical loads. In order to have a minimum base from which to start, this chapter assumes compliance with all basic requirements necessary to provide support for vertical loads and lateral loads other than earthquake. These basic requirements include, but are not limited to, provisions for the extent of investigation needed to establish criteria for fills, slope stability, expansive soils, allowable soil pressures, footings for specialized construction, drainage, settlement control, and pile requirements and capacities. Certain detail requirements and the allowable stresses to be used are provided in other chapters of the *Provisions* as are the additional requirements to be used in more seismically active locations.

7.2 STRENGTH OF COMPONENTS AND FOUNDATIONS

The resisting capacities of the foundations must meet the provisions of Chapter 7.

7.2.1 Structural Materials

The strength of foundation components subjected to seismic forces alone or in combination with other prescribed loads and their detailing requirements must be as determined in Chapters 9, 10, 11, or 12.

7.2.2 Soil Capacities

This section requires that the building foundation without seismic forces applied must be adequate to support the building gravity load. When seismic effects are considered, the soil capacities can be increased considering the short time of loading and the dynamic properties of the soil.

7.3 SEISMIC PERFORMANCE CATEGORIES A AND B

There are no special seismic provisions for the design of foundations for buildings assigned to Categories A and B.

7.4 SEISMIC PERFORMANCE CATEGORY C

Extra precautions are required for the seismic design of foundations for buildings assigned to Category C.

7.4.1 Investigation

The Regulatory Agency may require a formal foundation investigation and a written report. Potential site hazards such as slope instability, liquefaction, and surface rupture due to faulting or lurching as a result of earthquake motions should be investigated when the Regulatory Agency feels the size and importance of the project so warrants or when there may be reason to suspect such potential hazards. A summary of the various types of ground failure that may occur during an earthquake, including a general discussion of analysis techniques is presented by the Earthquake Engineering Research Institute (1986).

One potentially serious form of ground failure is liquefaction. Liquefaction can substantially reduce the shear strength of some types of soils, particularly loose, saturated sands. As a result of liquefaction, foundation soil bearing capacity and frictional capacity between the soil and the foundation can be reduced. A dramatic example of the effects of liquefaction on foundation soil capacity was the severe settlement and tilting of buildings that occurred during the 1964 earthquake in Niigata, Japan. Buildings supported on both spread footings and piles were affected. Other effects of liquefaction that have been observed during earthquakes include lateral spreads and flow slides, floating of lightweight embedded structures, and increased lateral pressures on retaining walls.

It is recommended that the soil engineer assess the potential for liquefaction during the geotechnical investigation for the building project. When a significant liquefaction hazard is found, it should be considered in arriving at an appropriate foundation solution, developing foundation design parameters, and evaluating performance.

Recent publications describing the liquefaction hazard and methods for evaluating it include those by Seed and Idriss (1982), Seed et al. (1983), National Research Council (1985), Koester and Franklin (1985), and Earthquake Engineering Research Institute (1986). Additional helpful references on this topic are cited at the end of this "Chapter 7 Commentary."

A widely accepted and economical method for evaluating liquefaction potential is the procedure developed by Seed and his co-workers (Seed et al., 1983) that relates liquefaction potential to the standard penetration resistance (SPT) blow count obtained in soil borings. This procedure is based on the actual performance of soil deposits during earthquakes and it makes use of the type of data that are usually obtained during the course of a foundation investigation for a building. An alternative approach is to perform undisturbed sampling and cyclic testing of liquefaction-susceptible soils (Seed and Idriss, 1982). Although this approach is appropriate in some situations, the former approach using SPT blow count generally is preferred because of the difficulties and expense involved in undisturbed sampling and cyclic testing of soils.

7.4.2 Pole-Type Structures

The use of pole-type structures is permitted.

7.4.3 Foundation Ties

One of the prerequisites of adequate performance of a building during an earthquake is the provision of a foundation that acts as a unit and does not permit one column or wall to move appreciably with respect to another. A common method used to attain this is to provide ties between footings and pile caps. This is especially necessary where the surface soils are soft enough to require the use of piles or caissons. Therefore, the pile caps or caissons are tied together with nominal ties capable of carrying, in tension or compression, a force equal to $A_v/4$ times the larger pile cap or column load.

A common practice in some multistory buildings is to have major columns that run the full height of the building adjacent to smaller columns in the basement that support only the first floor slab. The coefficient applies to the heaviest column load.

Alternate methods of tying foundations together are permitted (e.g., using a properly reinforced floor slab that can take both tension and compression). Lateral soil pressure on pile caps is not a recommended method because the motion is imparted from soil to structure (not inversely as is commonly assumed), and if the soil is soft enough to require piles, little reliance can be placed on soft-soil passive pressure to restrain relative displacement under dynamic conditions.

If piles are to support structures in the air or over water (e.g., in a wharf or pier), batter piles may be required to provide stability or the piles may be required to provide bending capacity for lateral stability. It is up to the foundation engineer to determine the fluidity or viscosity of the soil and the point where lateral buckling support to the pile can be provided (i.e., the point where the flow of the soil around the piles may be negligible).

7.4.4 Special Pile Requirements

Special requirements for concrete or composite concrete and steel piles are given in this section. The piles must be connected to the pile caps with dowels.

Although unreinforced concrete piles are commonly used in certain areas of the country, their brittle nature when trying to conform to ground deformations makes their use in earthquake-resistant design undesirable. Nominal longitudinal reinforcing is specified to reduce this hazard. The reinforcing steel should be extended into the footing to tie the elements together and to assist in load transfer at the top of pile to the pile cap. Experience has shown that concrete piles tend to hinge or shatter immediately below the pile cap so tie spacing is reduced in this area to better contain the concrete. In the case of the metal-cased pile, it is assumed that the metal casing provides containment and also a nominal amount of longitudinal reinforcement in the lower portion of the pile.

Bending stresses in piles caused by transfer of seismic motions from ground to structure need not be considered unless the foundation engineer determines that it is necessary. It has been a convenient analytical assumption to assume that earthquake forces originate in the building and are transmitted into and resisted by the ground. Actually the force or motion comes from the ground--not the structure. This makes the necessity of interconnecting footings more important, but what is desired is stability--not the introduction of forces.

Possibly the simplest illustration is shown in Figure C7-1. Consider a small structure subjected to an external force such as wind; the piles must resist that force in lateral pressure on the lee side of the piles. However, if the structure is forced to move during an earthquake, the wave motion is transmitted through the firmer soils, causing the looser soils at the surface and the building to move. For most structures, the structure weight is negligible in comparison to the weight of the surrounding surface soils. If an unloaded pile were placed in the soil, it would be forced to bend similar to a pile supporting a building.

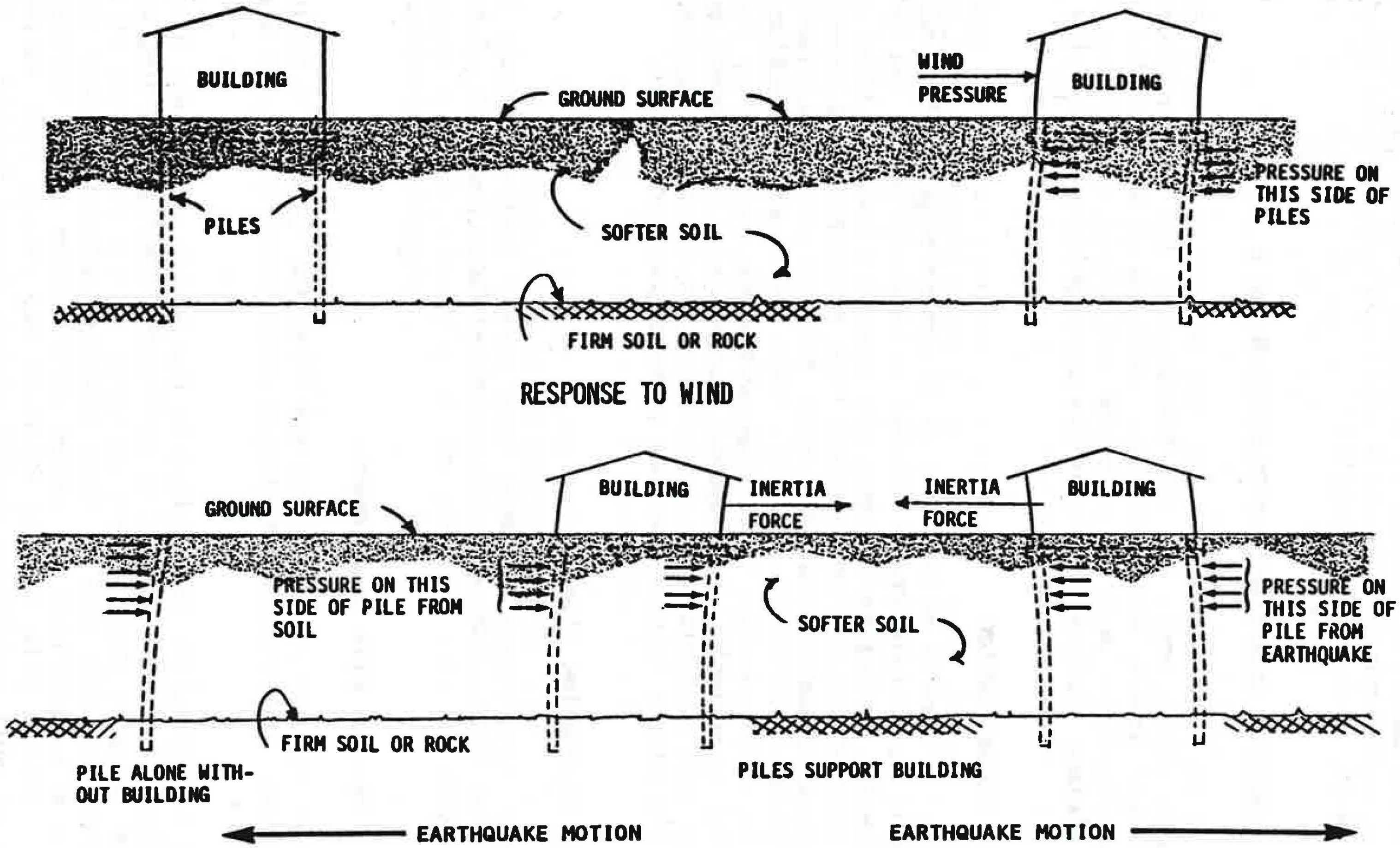

FIGURE C7-1
Response to earthquake.

The primary requirement is stability, and this is best provided by piles that can support their loads while still conforming to the ground motions and, hence, the need for ductility.

7.5 SEISMIC PERFORMANCE CATEGORIES D AND E

For Category D or E construction, all the preceding provisions for Categories A, B, and C apply for the foundations, but the earthquake detailing is more severe and demanding. Adequate pile ductility is required and provision must be made for additional reinforcing to ensure, as a minimum, full ductility in the upper portion of the pile.

7.5.1 Investigation

While the normal pressures on basement walls and retaining walls under normal or static conditions may be assumed to be predictable, the data for loads on walls during earthquakes are meager. Analyses based on the normal assumptions indicate rather high pressures, but general experience in earthquakes indicates that failures have not usually resulted. There is evidence, however, that under some conditions, especially in softer soils, these high pressures may occur. Consequently, after considering the size and importance of the project and the particular soil conditions, it is left for the foundation engineer to determine the design lateral pressure under dynamic conditions.

7.5.2 Foundation Ties

The additional requirement is made that spread footings should be interconnected by ties. The reasoning explained above under Sec. 7.4.3 also applies here.

7.5.3 Special Pile Requirements

Additional pile reinforcing over that specified for Category C buildings is required. The reasoning explained above under Sec. 7.4.4 also applies here.

Also, special consideration is required in the design of concrete piles subject to significant bending during earthquake shaking. Bending can become crucial to pile design where portions of the foundation piles may be supported in soils such as loose granular materials and/or soft soils that are susceptible to large deformations and/or strength degradation. Severe pile bending problems may result from various combinations of soil conditions during strong ground shaking. For example:

- Soil settlement at the pile-cap interface either from consolidation of soft soil prior to the earthquake or from soil compaction during the earthquake can create a free-standing short column adjacent to the pile cap.

- Large deformations and/or reduction in strength resulting from liquefaction of loose granular materials can cause bending and/or conditions of free-standing columns.

- Large deformations in soft soils can cause varying degrees of pile bending. The degree of pile bending will depend upon thickness and strength of the soft soil layer(s) and/or the properties of the soft/stiff soil interface(s).

Such conditions can produce shears and/or curvatures in piles that may exceed the bending capacity of conventionally designed piles and result in severe damage.

It is prudent to design piles to remain functional during and following earthquakes in view of the fact that it is difficult to repair foundation damage. The desired foundation performance can be accomplished by proper selection and detailing of the pile foundation system. Such design should accommodate bending from both reaction to the building's inertial loads and those induced by the motions of the soils themselves. Examples of designs of concrete piles include:

- Use of a heavy spiral reinforcement and

- Use of exterior steel liners to confine the concrete in the zones with large curvatures or shear stresses.

These provide proper confinement to ensure adequate ductility and maintenance of functionality of the confined core of the pile during and after the earthquake.

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Chapter 8 Commentary

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS

BACKGROUND TO ARCHITECTURAL CONSIDERATIONS

With respect to architectural systems, the primary intent was to investigate and develop seismic design standards for the performance of these building systems and their components of a building as they affect life safety. This "Chapter 8 Commentary" discusses the general attitudes and concepts adopted in approaching the subject. Of secondary but still critical importance was the examination of the damage control aspects of those critical facilities most necessary for the survival and recovery of the general public immediately following a major earthquake.

A methodology was devised to relate the following three basic items:

- Architectural Components--An orderly classification was established for architectural components and systems that encompasses broad general areas but is definitive enough to give guidance for similar conditions not specifically spelled out or covered.

- Occupancy Classification--Current building code occupancy classifications are based primarily on fire safety and as such do not necessarily or appropriately relate to seismic needs. Accordingly, provisions were developed to relate occupancy classification to the respective hazards of their seismic exposure. See Sec. 1.4.2 of the "Chapter 1 Commentary" for a detailed explanation.

- Performance Standards--It was deemed desirable to develop performance standards and not to rely on mathematical coefficients as has been the norm in standards of this type. For example, the design of a suspended ceiling in a hospital should have a higher level of performance capability than the same system in a warehouse in order to provide for life safety and maintenance of operability. On the other hand, for certain systems or components such as exterior wall panels, the concern for life safety requires similar performance of the system regardless of the occupancy involved. However, this objective could not be fulfilled and the end result is similar to the traditional approach using numerical factors.

The objective was to study the effects of seismically induced forces and deformations on the nonstructural (specifically, architectural) components in all types of building uses. Appropriate guidelines and design provisions for architectural systems and components were to be developed from a life safety standpoint. Each architectural component was to be examined as a function of expected performance, building occupancy and function, and its placement or location as a component of the building system. Finally, consideration was to be given to the architectural planning and design process as a means of improving the man-built environment from a life safety standpoint relative to seismic hazards.

The building designer has a responsibility to consider the relative levels of damage experienced by a building during an earthquake. These levels are a direct function of:

- The architectural concept as expressed by the design of the building,
- The resistance of the materials of construction, and
- The intensity of the ground motion.

The initial overall architectural concept has a direct bearing on the seismic resistance of a building and a considerable effect on the potential mitigation of hazards resulting from seismic forces. For the architect, certain principles and responsibilities hold just as true in designing systems and components for earthquake-resistant buildings as in the creation of any functional object. The designer, in addition to conceiving a rational design concept of the total building for seismic loading, must articulate all components into a logical system integrated as a unit rather than as a series of unconnected parts.

Architectural systems may be affected directly by the seismic forces or indirectly by interaction with the structural framing system or other architectural or mechanical and electrical systems. Fabrication methods used to connect the component parts to the structure or to each other are therefore as critical as the preliminary design. Attachment details therefore require specific attention since a dislodged roofing tile unit falling from a building could be as lethal to an individual as the failure of a primary girder. The life safety aspect of falling building debris associated with earthquake damage is related to a series of variables that include:

- The relationship of the location of the earthquake with respect to densely populated urban centers,

- The time of day (number of people in the area), and
- The design and construction characteristics of the building occupied by or immediately adjacent to people.

Depending on the time of day and the amount of activity without and within the building, falling debris from the building may cause as great a number of casualties to pedestrians or motorists as to building occupants. It was with such potential exterior hazards in mind that the City of Los Angeles enacted a "parapet ordinance" in 1949 that requires the strengthening or removal of hazardous parapets and appendages to buildings. The potential hazard was demonstrated during the 1971 San Fernando earthquake when the only fatality in downtown Los Angeles, approximately 20 miles from San Fernando, occurred when a pedestrian was struck by debris from a collapsing parapet of an unstrengthened old building.

BACKGROUND TO MECHANICAL AND ELECTRICAL CONSIDERATIONS

With respect to mechanical and electrical systems, the objective was to develop seismic criteria for the design and construction of these building systems and equipment and their attachments to the building structure so as to increase the protection of life and public welfare. A secondary objective was to define an acceptable level of damage. In so doing, consideration was given to the occupancy and function of the building.

Traditionally, mechanical and electrical systems for buildings have been designed with little, if any, regard to stability when subjected to seismic forces. Exceptions are to be found in nuclear power plant design and other special-purpose and high-risk structures. Equipment supports have been generally designed for gravity loads only, and attachments to the structure itself were often deliberately designed to be flexible to allow for vibration isolation or thermal expansion.

Few building codes, even in regions with a history of seismic activity, have contained provisions governing the behavior of mechanical and electrical systems. One of the earliest references to seismic bracing can be found in NFPA Pamphlet 13, *Sprinkler Systems*. This pamphlet has been updated periodically since 1876, and seismic bracing requirements have been included since about 1940. Until recently, few data were available regarding damage to mechanical and electrical equipment. Reports on the 1964 Alaskan earthquake and the 1971 San Fernando earthquake, however, document damage to mechanical and electrical systems and highlight the problem (Ayres et al., 1964 and 1972; Sharpe et al., 1972). These reports indicate that some buildings with only minor structural damage became uninhabitable due to failures of mechanical and electrical systems.

Mechanical and Electrical Considerations

As a result of the San Fernando earthquake, legislation establishing seismic criteria for health care facilities was passed in California (SB 519, 1972). This bill, which was in essence an extension of the *Field Act* (*California State Education Code*) to health care facilities, included for the first time seismic requirements for mechanical and electrical equipment and systems. The resulting regulations (*California Administrative Code*) apply to all health care facilities constructed in the state after April 1, 1974. The intent of the law is that those facilities that "must be reasonably capable of providing services to the public after a disaster" should be "designed and constructed to resist, insofar as practical, the forces generated by earthquake, gravity, and winds." The regulations require that mechanical and electrical systems be anchored so as to remain in place and be designed to remain operable after an earthquake. Another example of a code that was changed to include requirements for mechanical and electrical equipment is the April 1973 edition of the U.S. Department of Defense Tri-Service Seismic Design Manual (1973). This document was used in the development of the amplification factor used in the provisions of Chapter 8.

In assessing the level of "acceptable damage," secondary effects were considered to a limited extent. Fires and explosions resulting from damaged mechanical and electrical equipment represent secondary effects of earthquakes; these were not considered, however, except as covered under Sec. 8.3.5. Further, the potential danger of secondary damage from falling architectural and structural components (which could inflict major damage to adjacent equipment and render it unusable) should be carefully assessed by building designers.

These secondary effects can represent a considerable hazard to the building, its occupants, and its contents. Steam and hot water boilers and other pressure vessels can release fluids at hazardous temperatures. Hot water boilers operating above 212 °F pose a particular hazard since the sudden decrease in pressure caused by a rupture of the vessel can result in instantaneous conversion of superheated hot water to steam with explosive disintegration of the remainder of the vessel. Mechanical systems often include piping systems filled with flammable, toxic, or noxious substances such as ammonia or other refrigerants. Some of the nontoxic halogen refrigerants used in air conditioning apparatus can be converted to a poisonous gas (phosgene) upon contact with open flame. The hot parts of disintegrating boilers (e.g., portions of the burner, firebrick) are at high enough temperatures to ignite combustible materials with which they might come in contact.

It was concluded that, while secondary effects should eventually be included in building regulations, the provisions of Chapter 8 represent a sufficiently drastic departure from current design practices and the inclusion of secondary effects should be left for the future development of seismic code provisions. This basic philosophy underlies much of the assignment of performance levels to different occupancies.

DESIGN CONDITIONS

Four aspects of seismic safety were considered as follows:

- General life safety,
- Property damage affecting life safety,
- Functional impairment of critical facilities affecting post-disaster recovery (loss of utilities, elevators, life safety elements, etc.), and
- Safety of emergency personnel such as fire and rescue teams.

These four objectives are closely interrelated because property damage resulting from the consequences of an earthquake can be a definite cause of life loss. As in the case of fire, the relative hazards to life safety are also directly related to the occupancy load and the actual use of the building. The greater the occupancy load, the greater the potential life loss during an earthquake. An unoccupied building does not present a hazard to life safety within the structure during an earthquake.

Earthquake damage studies have shown that the placement of nonstructural elements on or in a building can significantly modify the seismic response of the structure. Heretofore this aspect of building design has received little attention. For example, prior seismic design philosophy implied that little structural damage should occur during moderate ground motion but some damage was expected to nonstructural components of the building. Thus, one could infer that as long as the possibility of structural collapse was minimal, there was little concern in design for earthquake-induced forces acting upon architectural and other nonstructural components. Recent earthquakes have demonstrated that the cost of damage to such components can be excessive.

Four sources of forces were considered with regard to the nonstructural components or systems:

- Seismic-induced forces acting directly on the component or system,
- Seismic-induced forces acting directly on the component or system joints or attachments,
- Seismic-induced deformation of the structural frame generating forces acting directly on the component or system, and
- Seismic-induced deformation of the structural frame generating

forces acting directly on the component or system joints or attachments.

SCOPE

In developing these provisions, it was necessary to analyze all nonstructural components for consequences to life safety and building function. Initially, all architectural components of a building were considered and those determined inconsequential to life safety were excluded. The remainder were assessed as to their potential effect on people and expected performance. The architectural components and systems considered were:

Building accessibility (including ground floor egress)
Exterior nonstructural walls (including parapets and large-scale veneers)
Veneers (small-scale ceramic mosaics, Venetian tile, etc.)
Canopies (except as means of egress)
Roofing units (tile, metal panels, slate, etc.)
Containerized and miscellaneous elements (planter boxes, etc.)

Fire detection systems
Fire suppression systems
Life safety communications systems
Smoke removal systems

Stairs
Elevators (operation only)
Vertical shafts (including elevator shafts)
Horizontal exits (only where otherwise required)
Public corridors
Private corridors

Full-height area and separation partitions
Full-height structural fireproofing
Full-height other partitions (including screens)
Partial-height partitions (including screens)

Ceilings, fire membrane
Ceilings, nonfire membrane

Equipment, ceiling mounted
Equipment, wall mounted
Equipment, freestanding unstable
Equipment, freestanding stable

Furniture, unstable
Furniture, stable

Art work, ceiling mounted
Art work, wall mounted
Art work, freestanding unstable
Art work, freestanding stable

This list includes most of the architectural components of a building that could present a hazard to the public. Similar listings were prepared for the mechanical and electrical components and systems. Initial consideration was given to 172 individual mechanical and electrical components in 37 occupancy classifications in an effort to arrive at common characteristics. Subsequently, these were consolidated, resulting in 19 component groups in the three seismic hazard exposure groups listed in Table 8-3. Although not all buildings contain all the components listed, the list does represent a fairly complete compilation of components and systems, some or all of which are usually present in typical or atypical buildings. Practical considerations--most notably enforcement--resulted in the modification, consolidation, and reduction in the number and type of components subject to seismic design requirements as specified in Tables 8-2 and 8-3. It is assumed that building designers will work as a team to provide for the required performance levels.

8.1-8.1.1 GENERAL REQUIREMENTS and INTERRELATIONSHIPS OF COMPONENTS

The general requirements establish minimum design levels for architectural, mechanical, and electrical systems and components recognizing occupancy use, occupant load, need for operational continuity, and the interrelation of structural and architectural, mechanical, and electrical components with the following exception: Those systems or components designated in Table 8-2 or 8-3 for L performance level that are in buildings assigned to Seismic Hazard Exposure Group I and are located in areas with the coefficient A_v less than 0.15 or that are in buildings assigned to Seismic Hazard Exposure Group II and are located in areas with the coefficient A_v less than 0.05 are not subject to the provisions of Chapter 8.

Seismic Hazard Exposure Groups are determined in Sec. 1.4 (Chapter 1). Multiple use requirements also are presented in that section.

The seismic force on any component shall be applied at the center of gravity of the component and shall be assumed to act in any horizontal direction. Vertical forces on architectural components are specified in Sec. 8.2.2. For vertical forces on mechanical and electrical components, see Table 8-3, Footnote b.

Although the components and systems included in Tables 8-2 and 8-3 are listed separately, significant interrelationships exist between them and should not be overlooked. For example, exterior, nonstructural, spandrel walls may shatter and fall on the streets or walks below seriously hampering accessibility and egress functions. Further, the rupture of one component could lead to the failure of another that is dependent on the first. Accordingly, the collapse of a single component may ultimately lead to the failure of an entire system. Widespread collapse of suspended ceilings and light fixtures in a building may render an important space or major exit stairway unusable.

Consideration also was given to the design requirements for these components to determine how well they are conceived for their intended functions. Potential beneficial and/or detrimental interactions with the structure were examined. The interrelationship between components or systems and their attachments were surveyed. Attention was given to the performance relative to each other of architectural, mechanical, and electrical components; building products and finish materials; and systems within and without the building structure. It should be noted that the modification of one component in Table 8-2 or 8-3 could affect another and, in some cases, such a modification could help reduce the risk associated with the interrelated unit. For example, landscaping barriers around the exterior of certain buildings could decrease the risk due to falling debris although this should not be interpreted to mean that all buildings must have such barriers.

The design of systems or components that are in contact with or in close proximity to structural or other nonstructural systems or components must be given special study to avoid damage or failure when seismic motion occurs. If a ceiling supports a wall, the intersection must be detailed to accommodate differential movements between them. Another example is where an important element of a system, such as a motor-generator unit for a hospital, is adjacent to a nonload-bearing partition. The failure of the partition might jeopardize the motor-generator unit and, therefore, the wall should be designed for a performance level sufficient to ensure its stability.

Where nonstructural wall systems may affect or stiffen the structural system because of their close proximity, care must be exercised in selecting the wall materials and in designing the intersection details to ensure the desired performance of each system.

8.1.2 Attachments

It is required that components be attached to the building structure and that all the required attachments be fully detailed in the design documents. These details should take into account the force levels and anticipated deformations expected or designed into the system. (See also Sec. 8.2.3.)

If an architectural component or system were to fail during an earthquake, the mode of failure would probably be related to:

- Faulty design of the component,
- Interrelationship with another component that fails,
- Interaction with the structural framing system,
- Deficiencies in its type of mounting, or
- Inadequacy of its attachments or anchorage.

The last is perhaps the most critical when considering seismic safety.

Building components designed without any intended structural function--such as in-fill walls--may interact with the structural framing system and be forced to act structurally as a result of excessive building deformation. The buildup of stress at the connecting surfaces or joints may exceed the limits of the materials. Spatial tolerances between such components thus become a governing factor. These provisions therefore emphasize the ductility and strength of the attachments for exterior wall elements and the interrelationship of elements.

Traditionally, mechanical equipment that does not include rotating or reciprocating components (e.g., tanks, heat exchangers) is rigidly anchored to the building structure. Mechanical and electrical equipment containing rotating or reciprocating components often is isolated from the structure by vibration isolators (rubber-in-shear, springs, air cushions). Heavy mechanical equipment (e.g., large boilers) is often not restrained at all, and electrical equipment other than generators, which are normally isolated to dampen vibrations, is usually rigidly anchored (e.g., switchgear, motor control centers). The installation of unattached mechanical and electrical equipment should be virtually eliminated for buildings covered by the provisions.

Friction cannot be counted on to resist seismic forces because it has been observed that equipment and fixtures often tend to "walk" due to rocking when subjected to earthquake motions. This is often accentuated by the vertical ground motions. Because frictional resistance cannot be relied upon, positive restraint must be provided for each system or component.

8.1.3 Performance Criteria

Each type of component or system subject to these provisions was evaluated as to its expected performance level. The goal of designing for several performance levels, which was established for initial guidance, is contained in Table C8-1 and C8-2. Levels of expected performance were assessed against levels of potential hazards to life safety according to the location and function of the component. Life safety was the overriding criterion for developing the levels of performance for each nonstructural component.

TABLE C8-1
Performance Criteria for Architectural Components and Systems

Matrix Letter Symbol	Ranking Performance Level No.	Performance Characteristic	Design Goal
S	1	Superior	Maximum resistance to lateral force design criteria; design limited to cosmetic damage; all operating functions to be unimpaired; minimize glass breakage (safety glass may crack); no loss of any fire rating or protection; system or component shall be able to handle 1.5 times the design deflections of any structural member to which it is attached or could have loads imposed on it due to structural member design movement.
G	2	Good	Average resistance to lateral force design criteria; no major fall-off of wall or ceiling components allowed; no glass fallout except for tempered glass fragments; all operating functions normally operable or readily repaired on site in a limited number of working days; fire ratings 75 percent (this does not mean 75 percent of unit is intact; it means that a 4-hour wall shall have 3-hour, etc.); minor damage to system or component structure is allowed; system or component shall be able to handle 1.0 times the design deflections of any structural member to which it is attached or could have loads imposed on it due to structural movement.
L	3	Low	Low resistance to lateral forces; glass fallout permitted; ceilings and lighting fixtures may fall down; major components must substantially stay in place but not operable until repaired; system or component structural damage may occur; fire ratings impaired; system or component shall be able to handle 0.5 times the design deflections of any structural member to which it is attached or could have loads imposed on it due to structural member design movement.
N	4	None	No performance standards required.

TABLE C8-2
Performance Criteria for Mechanical/Electrical Components and Systems

Performance Criteria Factor	Performance Level	Design Goal
1.5	Superior (S)	High resistance to static and dynamic seismic forces; all operating functions unimpaired; no broken piping regardless of size; no interruptions of utility services other than normal transfer functions to alternate sources.
1.0	Good (G)	Moderate resistance to static and dynamic forces; all major equipment normally operable or easily repaired on site; no broken main distributing piping or vessel; no shorted/broken electrical circuits.
0.5	Low (L)	Low resistance to static and dynamic seismic forces; major equipment must substantially stay in place; broken main distribution piping and vessels tolerated; fallout of lighting fixtures tolerated.
0.0	None (N)	No performance standards required.

NOTE: The design goals listed above do not represent absolute levels. The complexity of mechanical and electrical equipment, piping and duct systems, electrical distribution systems, etc., together with the unique magnitude and time spectrum characteristics of each seismic event make this impossible. It is believed that the above design goals are achievable and that equipment and systems designed to these criteria will result in an acceptable minimum percentage of failures and danger to the public.

Once a performance criteria is established for a component or system, it should be designed to operate or function at that level. Specifically, performance criteria are utilized to define standards against which expected performance is to be measured in terms of life safety.

The performance characteristic levels, P, given in Table 8-1 resulted from consideration of a combination of factors including performance and value judgment based on personal experience. In the development of the P values, the formulas utilizing this factor are based on broad assumptions. Therefore, the differences in performance levels are sizeable. It should be noted that 1.0 is considered the base performance value for most components.

The factor, P, is a dimensionless modifier of the design force level on a component or system based upon its interrelationship with Seismic

Hazard Exposure Group (occupancy or use group) for the building in which it is located. These are shown in Tables 8-2 and 8-3.

8.2 ARCHITECTURAL DESIGN REQUIREMENTS

8.2.1 General

The architectural design requirements provide that calculations, criteria, or other substantiation be prepared and included as part of the design documentation. The use of standard designs for certain building components, based upon conservative values for variables, may be applicable to most buildings.

The location of a building is important from three viewpoints:

- Site-related effects of ground-shaking including landslide and liquefaction,
- Relationship to densely populated areas, and
- Linkage to site plan.

Location and geographic distribution of buildings have a direct relationship to potential life loss. In areas of high-intensity ground-shaking, the possibility of significant failure of architectural and other nonstructural systems increases. While hazard to life safety within a building remains constant, potential life loss can be significantly increased if the building is also located in a densely populated urban area. The time of day also can be of importance because of the possibility of a large number of persons being inside or adjacent to the exterior of the building.

The placement of buildings on a site can significantly affect the impact that collapse, or failure, of architectural and nonstructural components can have on:

1. The entrance or egress of occupants to the building,
2. The blocking of streets, and
3. Accessibility to the building by fire and rescue teams.

Accordingly, guidelines were established to cover the respective hazards and their relationships to both interior occupancies and exterior circulation.

Many variables exist in building linkages to the site plan. Perhaps the most obvious constraint is the effect of lot size and/or location. Few options exist for either the architect or engineer to position buildings on small lots or restricted sites in congested urban centers. However, in the case of large building sites, such as those found in regional shopping complexes surrounded by large parking areas, hazard mitigation can be properly considered. For example, as noted previously, properly placed landscaping around the exterior of a building can provide a protective barrier from falling hazards. Accessibility to a damaged building for fire and rescue teams is essential and, therefore, the entrance and egress to the building should be protected. All space surrounding a building does not necessarily affect accessibility--only those areas that are associated with accessibility to the building site and entrance to and egress from each building.

Accessibility for Group III occupancies is most important. (See Sec. 1.4.2.5.) Experience has shown that access can be lost or seriously compromised by debris falling from both the building involved and adjacent structures. In order to assure that future improvements on adjacent property do not jeopardize this accessibility, the provisions require that adequate protection of such access be provided. The simplest means of resolving this adjacent property problem would be to restrict the location of the access to at least 10 feet from any adjacent property line. If there is an existing building on the adjacent property and it is, for example, constructed of reinforced masonry, the architect should seriously consider providing a greater degree of protected access. This would avoid the potential hazard that the existing adjacent structure may present. Although not covered by these provisions, the designer also should consider the possible loss of access along streets, highways, or bridges adjacent to the site.

8.2.2 Forces

The design seismic force is dependent upon the weight of the system or component, the seismic coefficient for the locality, the seismic coefficient for the component, and the required performance characteristic. The term A_v is a variable parameter dependent on local earthquake history and probability of occurrence. The maps in Chapter 1 specify values for locations across the United States. The performance characteristic relates to the occupancy group and the component or system involved per Table 8-2.

Certain design requirements for architectural components in areas of low seismicity are eliminated by the exceptions of this section. However, the designer may wish to provide for some increased safeguards in order to lessen the potential cost to his client for architectural components. This is not mandated in the provisions.

It should be noted that the minimum lateral design force usually specified for interior partitions (i.e., the 5 pounds per square foot criteria found in most codes) may exceed the forces developed from Eq.

8-1, thereby eliminating the need for design for seismic forces, but not detailing, of these walls.

The C_c factor in Table 8-2 was originally based on the use of the working stress design and was similar to the C_p factors specified in the *Uniform Building Code* and Title 24 of the *California Administrative Code*. In some cases, these values were modified slightly based upon experience and judgment. In the case of exterior nonbearing wall parapets, the C_c value was considerably reduced since the developers of the preliminary version (ATC 3-06) of the *Provisions* did not believe they could justify a difference between a parapet and a cantilever portion of an exterior wall. The poor history of unreinforced masonry parapets, which was the basis of prior high C_c values, should not be transferred to newer and properly designed systems.

When the decision was made to use stresses approaching yield in the provisions, the C_c values were modified so as to be in accordance with these higher allowable stresses; the final proposed C_c factors (and existing code C_p factors) are somewhat arbitrary and, consequently, need continued review and further research. It is hoped that future investigations will distinguish between a failure to meet the numerical coefficients of a standard and a failure based on noncompliance with the basic intent of a standard and will thereby result in more rational values of these factors.

The modifications that resulted in the C_c values presented in Table 8-2 were developed from comparative computations and application of subjective judgment.

From prior codes:

$$F_p' = ZC_p'W_p, \tag{C8-1}$$

and from Eq. 8-1:

$$F_p = A_v C_c P W_c, \tag{C8-2}$$

where F_p' = the force at working stress level, F_p = the force at yield, Z = the seismic zone factor, A_v = the effective peak velocity-related acceleration coefficient, C_p' = the prior component factor, C_c = the new component factor, W_p and W_c = the weight of component, and P = the performance factor. The ratio of yield stress to working stress has been accepted as approximately 1.60. In working stress design, there is an allowance for a design stress increase of 33 percent for seismic loadings. These factors have been combined to develop the relation:

$$F_p = (1.6/1.33)F_p' \text{ or } 1.2\ F_p', \tag{C8-3}$$

For example, assuming $Z = 1$, $A_v = 0.4$ and $P = 1$, then:

$$F_p = 1.2[(1)(C_p')(W_p)], \qquad \text{(C8-4)}$$

and

$$F_p = 0.4\ (C_c W_c). \qquad \text{(C8-5)}$$

If $C_p' = 0.2$ for a partition, and taking $W_c = W_p$ then:

$$1.2(0.2\ W_c) = 0.4\ C_c W_c \qquad \text{(C8-6)}$$

and

$$C_c = 0.6. \qquad \text{(C8-7)}$$

The amplification effects due to building height were not considered significant because of the manner in which the values were assigned to C_c and P, the relatively light weight of typical components or systems (as compared to the building weight), and the desire to maintain a simple form for Eq. 8-1.

8.2.3 Exterior Wall Panel Attachment

This section requires ductility and rotational capacity for exterior panels. To ensure that the attachment is ductile, care must be taken in its detailing. To minimize the possibility of a brittle-type failure, the attachments to the structural frame must be designed to accommodate (by bending or rotation) the potential differential motions between the component and the structural frame.

8.2.4 Component Deformation

Earthquake motions induce deflections at each floor level. The difference in the deflections of the top and bottom of each story is the story drift. Walls, partitions, glazing, etc., in each story of a building must be capable of accommodating the story drift without causing a life safety hazard. The larger story drifts resulting from the inherently more-flexible steel or reinforced concrete moment frame buildings may cause damage to floor-to-floor partitions and other nonstructural systems (e.g., stairs, elevator shafts) unless proper design considerations are provided. Such nonstructural damage as evidenced in past earthquakes can exceed 50 percent of the replacement value of a building and can also endanger the occupants. In comparison, shear wall buildings are usually more rigid than moment frame structures and therefore have smaller story drifts.

Architectural design considerations must take into account the components of deformation that can occur:

- Direct deformation in the component or system itself,
- Direct deformation in the joints or attachments of the component or systems,
- Deformation of the component or system produced by structural frame or structural wall movements,
- Deformation in the joints or attachments of the component or system produced by structural frame or structural wall movements.

The drift values to be considered in the design of components are those derived in Sec. 4.6.1. These values can be reduced by one-half for components with a required performance characteristic level of L.

All architectural systems or components attached to or framed within the structural system must be capable of accommodating a story drift of Δ without failure or should be separated from the structure to prevent the deformations of the structure from affecting the architectural system or component. Such isolation can be accomplished by providing a degree of separation at least equal to the calculated drift from Sec. 4.6.1. Rigid elements (e.g., stairways, masonry walls) should be given special consideration since not only are they subject to damage and loss of function from structural deformations but also, of equal importance, their stiffness may significantly affect the structural system to which they are connected. In each instance both structural and fire resistance requirements have to be reconciled.

Differential vertical movement between horizontal cantilevers in adjacent stories (i.e., cantilevered floor slabs) has occurred in past earthquakes. The possibility of such effects should be considered in the design of exterior walls.

8.2.5 Out-of-Plane Bending

Most walls are subject to out-of-plane forces when a building is subjected to an earthquake. These forces and the bending they induce must be considered in the design of wall panels. This is particularly important for systems composed of brittle materials and/or low flexural strength materials. The conventional limits based upon deflections as a proportion of the span may be used with the applied force as derived from Eq. 8-1 and Table 8-2.

8.2.6 Raised Access Floors

ANSI A58.1-1982 may serve as a guide for the weights supported by raised access floors.

8.3 MECHANICAL AND ELECTRICAL DESIGN REQUIREMENTS

8.3.1 General

The mechanical and electrical design forces are assumed to be imposed from any horizontal direction. The vertical forces as noted in Footnote b of Table 8-3 are assumed to be one-third of the maximum horizontal forces. The designer is allowed an option of justifying a reduction in the seismic forces required by this chapter. Such justification may be made by performing a dynamic analysis based upon established principles of structural dynamics.

8.3.2 Forces

Equation 8-2 should be used for the design of components and their attachments. The method of attachment for mechanical and electrical components must be either by fixed or direct attachment to the building or by attachment with a resilient mounting system. Reliance on friction to resist seismic forces is not permitted.

If an item of mechanical or electrical equipment is rigidly anchored to the building structure, seismic forces are transmitted directly to the equipment. The design force is dependent on the performance rating assigned to the particular piece of equipment.

Where fixed (rigid) attachments are used for components with performance levels of S or G in areas with coefficient A_v equal to or greater than 0.15, certification must be obtained from the component manufacturer that the component is capable of withstanding the design forces without sustaining damage. Shaking-table tests or three-dimensional shock tests may be used for certification if an analysis is too difficult to perform. Components frequently can withstand considerable force in one horizontal direction but may fail if a concurrent force is applied from another horizontal direction.

Mechanical equipment such as reciprocating or rotating machinery traditionally has been mounted on resilient mounting systems, particularly when installed on upper floors of structures. The primary reason for this type of mounting system is to dampen or isolate the vibration emanating from the equipment and thereby inhibit sound and vibration transmission through the building structure.

The structural system and the resilient mounted equipment form a complex dynamic system. To account for this, the amplification introduced by the relationship of the equipment support period and the building period should be included if the equipment is to survive the earthquake as required for S or G performance criteria levels. It is recognized that a rigorous solution of this problem requires a detailed computer dynamic analysis. The designer is given the option of making a rigorous dynamic analysis of the equipment and its supporting system by established principles of structural dynamics to qualify the equipment. As an alternate, the Tri-Service Seismic Design Manual includes a method based on an approximation of the system as a single-degree-of-freedom system. This method was adapted to be the general methodology followed in these provisions as one method of qualifying the equipment. An attempt was made to determine whether techniques are available at present to conduct a meaningful dynamic analysis of elastic restraining systems. The state-of-the-art appears to be as follows:

- Only one commercially available computer program is known to be available that provides a form of dynamic analysis of elastic restraining systems. Because of the absence of actual earthquake data, this program makes assumptions regarding frequency components and their duration and limits itself to frequencies in the range of 0.1 to 16 Hz. The program was developed by the California Institute of Technology for a manufacturer of resilient support systems and access is available only through that manufacturer.

- There are sensors and recording systems available that can measure and record directly on magnetic tape the various parameters during a seismic event. The data could form the basis for an improved dynamic analysis program and make possible improved design techniques for resilient mounting systems.

- There is a need for the installation of full dynamic response sensors at existing strong-motion instrumentation stations. There is also a need for the development of adequate computer programs that can be made available to all qualified designers in this field.

Resilient mounting attachments must be designed to decelerate movement of the component or system at a rate that will not generate forces in excess of those calculated from Eq. 8-2. The resilient mounting systems can include such items as stable springs, pneumatic restraining devices, or elastic restraining devices; however, any device used must be capable of withstanding the forces determined from Eq. 8-2. It was decided that the equation for calculating the seismic forces on mechanical and electrical equipment should include two variable parameters in addition to those required in Sec. 8.2. Therefore, two additional factors--a_c (an amplification factor for resiliently mounted equipment) and a_x (an amplification factor to increase the applied forces dependent on the height of the equipment in the building)--are included in Eq. 8-2. The values of the various factors and coefficients were determined as indicated below.

C_c Factor Determinations

Initially, C_c was defined as:

$$C_c = a/g, \tag{C8-8}$$

where g = acceleration due to gravity (ft/sec^2) and a = estimated design acceleration (ft/sec^2). The quantity "a" represented an amplification of the effective peak acceleration coefficient for the coefficient A_v equal to or greater than 0.20. The amount of amplification was related to similar factors in the California regulations (SB 519, 1972). In order to bring C_c into conformance with other sections of the provisions, the concept was changed to define C_c as a numerical dimensionless factor related to the mechanical and electrical components in Table 8-3. The numerical values as shown in Table 8-3 were developed by using an analogy to the C_p values in Table T17-23-3 of Title 24 of the California Administrative Code as indicated below.

From Title 24:

$$F_{p'} = C_p{}' W_p P, \tag{C8-9}$$

where

$F_p{}'$ = the design force,

$C_p{}'$ = the Cp value from Table T17-23-3, and

Wp = weight of component

and from Eq. 8-2:

$$F_p = A_v C_c a_c a_x W_c P, \tag{C8-10}$$

where

F_p = the design force,

A_v = Effective Peak Velocity-Related Acceleration Coefficient (EPV),

a_x = 1 (for comparison purposes),

a_c = 1 (for comparison purposes),

$W_c = W_p$ = weight of component, and

P = 1.5 (for a hospital).

F_p was set equal to 1.2 Fp′ because the design in these provisions is based on yield strength and not on working strength as in Title 24. Thus:

$$A_v C_c a_x a_c W_c P = 1.2\ C_p{}' W_c. \qquad \text{(C8-11)}$$

Substituting $A_v = 0.40$ and cancelling W_c:

$$0.4\ C_c A_x A_a\ 1.5 = 1.2\ C_p{}' \qquad \text{(C8-12)}$$

or

$$C_c = 2.0\ C_p{}'/A_x A_c \qquad \text{(C8-13)}$$

Table T17-23-3 prescribes $C_p' = 1.0$ for essential mechanical equipment, and, thus, $C_c = 2.0$ for comparable mechanical and electrical components with an S performance level. Values for other equipment were then scaled to the above.

Structure Amplification Factor (a_x)

The use of the building amplification factor a_x required similar considerations to those above. A review of the literature (U.S. Department of Defense, 1974; Fagel et al., 1973) as well as a desire to motivate designers to locate heavy mechanical or electrical equipment in the lower levels of the building prompted the use of such a factor. One method of accounting for this effect is to use a formula based on the distribution factor C_{vx} from Eq. 4-6a. The use of this formula requires cross-referencing to Chapter 4 and involves concepts that may be unfamiliar to mechanical and electrical engineers. In addition, it tends to result in values in excess of those considered reasonable. Therefore, it was decided to use an approach derived from information contained in the Tri-Service Manual but differing from it as follows: The equation used in the Tri-Service Manual gives directly the acceleration due to seismic forces (as a fraction of gravity) at each level of the building. This number is then combined with a soil constant such that the product of the structure amplification factor and the soils constant ($A_h C_s$) represents a number comparable to the product of the EPV coefficient (A_v), the C_c factor, and the structure amplification factor ($A_v C_c a_x$).

It was judged that a 100 percent increase for the top level of the building was reasonable and, thus, $A_x = 1 + (h_x/h_n)$.

Equipment Amplification Factor (a_c)

A relationship for determining this amplification factor was developed by assuming that the response of the building at the equipment level can be approximated by a sinusoidal loading of the form P sin(ωt). The amplification factor for this type of motion is then related to the acceleration resulting from the increase in the equipment response due to the building response. Whenever the period of the building and that of the equipment are approximately equal, resonance occurs. The equation is based on the theory of harmonic motion (Timoshenko and Young, 1955) and is used to compute the amplification factor:

$$a_c = 1/\sqrt{[1 - (\omega/\omega_a)^2]^2 + [2\lambda\omega/\omega_a]^2}, \qquad \text{(C8-14)}$$

where a_c is the amplification factor, ω is the natural frequency of the equipment (rad/sec), and ω_a is the natural frequency of the structure (rad/sec), and λ = the percent of critical damping of equipment.

The Tri-Service Manual has selected a value of λ equal to 2 percent. Substitution of the value $2\pi/T$ for ω, and $2\pi/T_c$ for ω_a produces the curve shown on Figure C8-1 which indicates a magnification factor of 25 at resonance. This was reduced to a factor of 2 for period rates between 0.6 and 1.4 seconds with all other period ratios having a factor of 1 for the following reasons:

- The damping coefficient λ is not constant at 2 percent during a seismic event.
- The building period is also not a constant because of deformation of the structure.

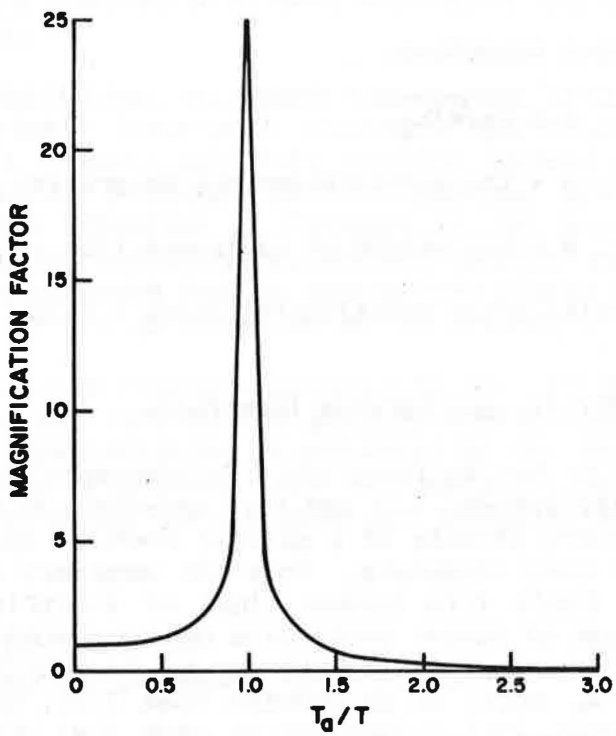

FIGURE C8-1
Magnification factor versus period ratio.

- The magnification factor graphs in the Tri-Service Manual are based on an approximation of the system as a single-degree-of-freedom type system. This is not considered to be representative of actual conditions. It should be noted, however, that period ratios in the range of 0.8 to 1.2 may result in considerably higher magnification and this must be considered in the design.

Component Attachment Period (T_c)

Equation 8-4 is derived from the basic mass response equation (Tri-Service Manual):

$$\omega = \sqrt{K/M_{me}}, \tag{C8-15}$$

where

ω = the circular frequency (rad/sec),

M_{me} = the mass of mechanical or electrical equipment ($lb\text{-}sec^2/in.$), and

$T = 2\pi/\omega$ (sec).

Combining the above equations:

$$T = 2\pi\sqrt{W/K_g}, \tag{C8-16}$$

where

g = the acceleration due to gravity ($in./sec^2$) and

W = the weight of equipment (lb).

Equation 8-4 results after substituting $2\pi/\sqrt{g} = 0.32$.

8.3.5 Utility and Service Interfaces

Special hazards to the building and its occupants are created by the failure of utility systems. It was felt necessary to give some consideration to secondary effects of a seismic event as an exception to the general rule followed elsewhere. Possible secondary effects are leakage of fossil fuels from broken lines or electrical short-circuit currents in excess of normal protective device capabilities. For this reason, for Seismic Hazard Exposure Groups II and III in areas with the coefficient A_v equal to or greater than 0.15, protective devices are required that will automatically stop fuel flows or interrupt current in the event earthquake motions greater than a designated intensity occur. Interruption of gas or high temperature energy supplies to buildings can be accomplished by installing seismic valves at the service connection to a building. Interruption of electrical service can be achieved by shunt-tripping the main circuit breakers when activated by a sensor that can detect excessive ground motion.

The rapid growth of urban electric distribution networks also was considered in development of the provisions. In many instances, utility companies have increased their distribution networks such that the fault current potentials that existed when a building was originally constructed have increased manyfold. This is particularly the case in urban areas where secondary network concepts are utilized. These networks, by adding transformer capacity, have reduced the reactance needed to limit fault current. In some cases, electrical facilities initially providing less than 25,000 amperes interrupting current now exceed 200,000 amperes or more, and incoming service equipment and distribution equipment within the structure are inadequate to handle such loads. This problem is of concern because phase-to-phase or phase-to-ground faults can develop during a seismic event in equipment not adequately designed and could completely consume the service entrance equipment, service protection equipment, and distribution equipment and represent a significant source of fire. The potential energy release of these fault currents is such that 1/4 in. by 4 in. cross-section bus bars, utilized in switchboards singly or in multiples, would melt as if in an electric arc furnace, and the molten copper would flow along the floor igniting any combustible material it encountered. The resolution of this problem is not within the scope of these provisions.

For essential facilities, equipment and systems requiring an S performance characteristic level must remain in operation after the disaster. For this reason, auxiliary on-site mechanical and electrical utility sources, or secondary utility sources, are recommended. No reference to this situation is included in the provisions because in most cases existing building regulations usually contain such provisions. It is recommended that an appropriate clause be included if the existing codes for the jurisdiction do not presently provide for it.

Sec. 8.3.5.2 requires flexible connections for utilities at the interface of movable portions of the structure to accommodate anticipated displacement. Base isolation is addressed by the *Provisions* only as a warning. It affects architectural and mechanical/electrical fittings only in a limited manner (where water and energy lines pass through the interface).

CHAPTER 8 TABLES:
OCCUPANCY-COMPONENTS-PERFORMANCE RELATIONSHIPS

The definitions of architectural components and systems, occupancy group types (Tables C8-3 and C8-4), and criteria for performance standards (Tables C8-1 and C8-2) were discussed above. It is apparent that interrelationships exist between the items and have a direct impact on

TABLE C8-3
Initial General Grouping of Occupancies

Group Letter	Classification	Subgroup Code No.	Occupancy Description
A	Typical public		Load of 100 or more (including assembly drinking/dining establishments)
B	Special public assembly	1	Open air only (not covered by roof)--stadiums, reviewing stands, park structures, etc.
		2	Regional shopping centers with enclosed shopping malls
C	Education (campus operations only; does not include 1 to 3 room adult school operation)	1	50 or more persons through 12th grade
		2	Less than 50 persons through 12th grade
D	Confined facilities	1	Mental, jails, prisons, restrained inmates
		2	Nurseries for child care only, nonambulatory
		3	Nursing homes, child care of kindergarten age or over, ambulatory
		4	Hospitals
E	Hazardous storage and facilities	1	Hazardous/flammable storage
		2	Less hazardous/flammable storage
		3	Woodworking, shops, factories; loose combustible fibers/dust
		4	Repair garages
		5	Aircraft repair hangers
F	General	1a	Regular gas/service stations, commercial nonvital vehicle storage garages
		1b	Storage/parking of emergency vehicles (e.g., ambulances, utility trucks)
		2a	Wholesale stores, general warehouses
		2b	Retail stores (including drinking/dining establishments with a load of under 100)

TABLE C8-3 Continued

Group Letter	Classification	Subgroup Code No.	Occupancy Description
		2c	Office buildings, low rise, up to 75-ft height
		2d	Office buildings, high rise, over 75-ft height
		2e	Print shops, factories, industrial plants
		2f	Police/fire stations, communication centers
		2g	Warehouses, emergency supplies storage (e.g., medical, food, chemicals)
		3	Aircraft hangers, open parking garages
G	Special facilities (including existing low fire hazard)	1	Ice plants, factories, workshops using noncombustibles, nonexplosives
		2	Lifeline facilities, utilities, power plants
H	Hotel/apartment houses	1	Hotels, convents, monasteries
		2	Apartments, low rise, up to 75-ft height
		3	Apartments, high rise, over 75-ft height
I	Dwellings	1	Dwellings, lodging houses, sheds
		2	Fences over 6-ft height, tanks, towers

NOTE: This initial grouping was developed using the 1973 *UBC* as a point of departure; modifications and additions were made to occupancy group types.

TABLE C8-4
Consolidated Occupancy Grouping[a]

Group	Description
III	Buildings housing critical facilities necessary to post-disaster recovery and requiring continuous operation during and after an earthquake. The terms "critical facilities" and "emergency" are defined as meaning designated by the governmental entity having jurisdiction. Examples are fire and police facilities, hospital facilities with emergency treatment facilities, emergency preparedness and communications centers, power stations and other utilities required as emergency facilities.
II	Buildings housing dense occupancies having a high transient population and/or sleeping conditions or critical facilities requiring operation in the immediate post-disaster period; restricted movement facilities. Examples are public assembly for 100 or more persons; open air stands for 2,000 or more persons; day care; schools; colleges; retail stores with more than 5,000 sq. ft floor area/floor or more than 35 ft in height; shopping centers with covered malls over 20,000 sq. ft gross area excluding parking; office buildings with more than 10,000 sq. ft/floor or over 4 stories in height; hotels, wholesale stores, apartments, factories, and printing plants over 4 stories in height; hazardous occupancies consisting of flammable or toxic gases or flammable or toxic liquids including storage facilities for same.
I	Low-density occupancies and generally low transient population. Examples are aircraft hangers; working facilities; factories, warehouses, printing plants, hotels, and apartment houses 4 stories or less in height; repair and storage garages; service stations; ice plants; single- and two-family dwellings; townhouses; retail stores less than 5,000 sq. ft/floor and 35 ft or less in height; public assembly for less than 100 persons; offices less than 10,000 sq. ft/floor or 4 stories in height.
Multiple Occupancy Structures	Due to the high cost of construction, travel and land, it is likely that shopping, living, entertainment, medical, and working facilities will be combined and designed into a single structure. Any "preconceived boxes" or occupancy classifications within which buildings are classified must be designed to take into consideration the possibility of such multiple occupancy structures. Some of the new convention centers and regional shopping center malls are in this category and represent a high-occupancy risk situation. In this case, it was concluded that the architectural systems and components are even more critical than in conventional buildings. Egress and accessibility to these structures are most important.

[a]Note that these groupings have been revised for the 1988 Edition; see Sec. 1.4.2.

the levels of life safety to be achieved. For example, a heavy piece of ceiling-mounted mechanical equipment presents a minimal hazard to life safety when located in a private garage whereas the hazard from such equipment increases significantly if it is located in a large hall for public assembly with a potential occupancy of more than 1,000. The hazard would be further increased if the attachment or mounting for the equipment was poorly designed. An additional increase in the hazard potential would occur if it was mounted on the ceiling of a hospital ward used 24 hours a day. As described above, the introduction of landscaped barriers may alter the life safety risk from falling objects. Accordingly, design trade-offs between variables could raise or lower the life safety hazard. Following this principle, the methodology for dealing with a set of variables was established.

Some critical variables affecting life safety that were used in this methodology are:

- Occupancy density;
- Building height;
- The need for functioning after an earthquake considering the overall occupancy critical use factor, the specific component use factor, the need for egress after an earthquake, and the need for functionability of fire protection;
- Adequate access for emergency personnel;
- Public hazard exposure outside the building;
- Critical exposure to major secondary hazards (e.g., fire, explosion);
- Familiarity of occupants with surroundings;
- Restriction on movement of occupants;
- Probable age and mobility of occupants; and
- Siting of the building.

Table C8-5 displays the initial results of the methodology when applied to measurement of the three basic variables. It presents these results in the form of a table labeled "Tentative Matrix." The variables are measured against each other and are subject to modification when other sets of variables are introduced. Application of the "Tentative Matrix" to any one architectural component and system correlates the element (subject to further modification if desired) to performance standards and occupancy group. Other patterns may be found by seeking relationships between the architectural component and its performance to occupancy group, or occupancy group and architectural component to performance standard. Thus, for most desired information, the "Tentative Matrix" display could be utilized to obtain correlation with performance standards, architectural element definition, or occupancy group type. The higher the performance standard displayed on the "Tentative Matrix," the higher the hazard posed by the architectural element in context with occupancy group characteristics. In this way, minimum force levels were developed.

In development of the *Provisions*, it became evident that a system was needed to measure all variables, to establish priorities for dealing with them, and to identify the interrelationships between all items and correlate their diverse characteristics. This early matrix is included in this commentary to facilitate evaluation of the method used and and to provide guidance for future work.

RELATED CONCERNS

Maintenance

Mechanical and electrical devices installed to satisfy the requirements of these provisions (e.g., resilient mounting systems or certain protecting devices) require maintenance to ensure their reliability and provide the protection for which they are designed in case of a seismic event. Specifically, rubber-in-shear mounts or spring mounts (if exposed to weathering) will deteriorate with time and, thus, periodic testing is required to ensure that their damping action will be available during an earthquake. Pneumatic mounting devices and electric switchgear must be maintained free of dirt and corrosion. How a Regulatory Agency could administer such periodic inspections was not determined and, hence, provisions to cover this situation have not been included.

Minimum Standards

Criteria represented in the provisions represent minimum standards. They are designed to minimize hazard for occupants and to permit, insofar as practicable, the continued functioning of facilities required by the community to deal with the consequences of a disaster.

They are not designed to protect the owner's investment, and the designer of the facility should review with the owner the possibility of exceeding these minimum standards so as to limit his economic risk.

The risk is particularly acute in the case of sealed, air-conditioned buildings with L performance levels where downtime after a disaster can be materially affected by the availability of parts and labor. The parts availability may be significantly worse than normal because of a sudden increase in demand. Skilled labor may also be in short demand since available labor forces may be diverted to high priority structures requiring repairs.

Architect-Engineer Design Integration

The subject of an architect-engineer design integration is being raised because it is believed that all members of the profession should clearly understand that Chapter 8 is a compromise based on concerns for enforcement and the need to develop, in what was a limited time frame, a simple, straightforward approach. It is imperative that from the outset architectural input concerning definition of occupancy classification and the required level of seismic resistance be properly integrated with the approach of the structural engineer to seismic safety if the design profession as a whole is to make any meaningful impact on the public conscience in this issue. Accordingly, considerable effort was spent in this area of concern. It is hoped that as the design profession gains more knowledge and sophistication in the use of seismic design, it will collectively be able to develop a more comprehensive approach to earthquake design provisions.

TABLE C8-5 Tentative Matrix

OCCUPANCY	COMPONENTS															
	Building accessibility	Ext. non-struct. walls	Veneers - small scale	Canopies	Roofing units	Containerized and miscellaneous elements	Fire detection	Fire suppression	Life safety communication	Smoke removal	Stairs	Elevators	Vertical shafts	Horizontal exits	Public corridors	Private corridors
	1.1	2.1	2.2	2.3	2.4	2.5	3.1	3.2	3.3	3.4	4.1	4.2	4.3	5.1	5.2	5.3
A. ASSEMBLEY																
1. Public Assembly - 100 or more	S	G2	G1	G1	G2	G	G	S	S	S	S	L3	L3	S	G	L
B. SPECIAL ASSEMBLY																
1. Open Air Stands	S	G2	G1	G1	G	G	G	G	S	L	S	L	L3	S	G	L
2. Shopping Centers	G	G2	G1	G1	G2	L	S	S	G	G	G	H3	L3	G	G	L
C. EDUCATION																
1. K through 12 - 51 or more	S	G1	G1	G1	G1	G	S	S	S	S	G3	L3	L3	S	G	G
2. K through 12 - 50 or less	G					L	G		S					G		G
3. Day Care						G	S		S					S		G
4. College - 51 or more						L	G		G					G		L
5. College - 50 or less	G	G1	G1	G1	G1	L	G	S	G	S	G3	L3	L3	G	G	L
D. CONFINED FACILITIES																
1. Restrained	G	G	G	L	L	L	S	S	S	S	G2	G2	L3	G	G	G
2. Nurseries - non-ambulatory	G	G		L	L	L			S		G2	G2	L3	G	G	G
3. Ambulatory	G	G		L	L	L			G		G2	G2	L3	G	G	L
4. Hospitals	S	S	G	G	G	G	S	S	S	S	S	S	S	S	S	S
E. HAZARDOUS																
1. Hazardous & flammable storage	G	L4	L	L	H	H	G	S	S	G	G2	L3	G2	S	G	L
2. Less hazard. & flam. storage	G								S	G			L3	G	L	H
3. Wood working factories	G								S	G						
4. Repair garages	L								G	L	G2					
5. Aircraft	L	L4	L	L	H	H	G	S	G	L	G	L3	L3	G	L	H
F. GENERAL COMMERCIAL																
1a. Service stations & nonvital vehicle storage garages	L	L5	L5	L5	L1	H	L	S	H	S	G	L	L3	G	G	L
1b. Storage & parking of emergency vehicles	S	G	G	G	L1	L	G	S	S	S	G2	L3	L3	G	G	G
2a. Wholesale stores & general warehouses	G	L5	L5	L5	L	L	L	G	G	L	G	L3	L3	G	L	H
2b. Retail stores, incl. drinking & dining under 100	G	G	G	G	G	G	G	S	S	G	G3	G	G		G	L
2c. Office bldgs. - low rise	G	G	G	G	G	G	G		G	G	G	G6	L3		G	G
2d. Office bldgs. - high rise	S	G	G	S	G	G	S		S	S	S	G	S	G	S	G
2e. Print shops, factories, industrial plants	G	L5	L5	L5	L	L	G		L	G	G	G	L3	G	G	L
2f. Police & fire stations, communication centers	S	S	S		G	G	S	S	S	S	S	S	S	S	S	G
2g. Warehouse (emergency supplies storage)	S	G	G1	G1	G	L	G	S	G	G	G	G	G	G	L	H
3. Aircraft hangers, open parking garages	L	L5	L	L5	L	L	L	G	L	L	G	L	L3	L	G	L

TABLE C8-5 continued

OCCUPANCY	Full height area and separation partitions	Full height structural fireproofing	Full height other partitions	Partial height partitions	Ceilings - fire membrane	Ceilings - non-fire membrane	Equipment - ceiling mounted	Equipment - wall mounted	Equipment - free standing unstable	Equipment - free standing stable	Furniture - unstable	Furniture - stable	Art work - ceiling mounted	Art work - wall mounted	Art work - free standing, unstable	Art work - free standing, stable
	COMPONENTS															
	6.1	6.2	6.3	6.4	7.1	7.2	8.1	8.2	8.3	8.4	9.1	9.2	10.1	10.2	10.3	10.4
A. ASSEMBLEY																
1. Public Assembly - 100 or more	G	S	L	L	S	G	G	G	G	L	G	L	G	G	G	L
B. SPECIAL ASSEMBLY																
1. Open Air Stands	G	G	L	L	G	L	G	G	G	L	G	L	G	G	G	L
2. Shopping Centers	G	G	G	L	G	G	G	L	G	N	L	N	G	L	G	N
C. EDUCATION																
1. K through 12 - 51 or more	G	G	G	G	GS	G	G	G	G	L	L	N	G	G	G	L
2. K through 12 - 50 or less	↑	↑	L	L	↑	L4	↑	L	↑	N	N	↑	↑	L	↑	N
3. Day Care	|	|	G	G	|	G	|	G	|	L	L	|	|	G	|	L
4. College - 51 or more	↓	↓	L	L	↓	L4	↓	L	↓	N	N	↓	↓	L	↓	N
5. College - 50 or less	G	G	L	L	G3	L4	G	L	G	N	N	N	G	L	G	N
D. CONFINED FACILITIES																
1. Restrained	G	G	G	L	G	L4	G	G	G	L	L	N	G	L	L	N
2. Nurseries - non-ambulatory	G	G	↑	G	G	L4	G	G	G	L	G	N	↑	↑	G	N
3. Ambulatory	G	G	↓	L	G	L4	G	G	G	L	L	N	↓	↓	L	N
4. Hospitals	S	S	G	G	S	G	S	S	S	S	G	L	G	L	G	L
E. HAZARDOUS																
1. Hazardous & flammable storage	S	S	L	N	S	L	L	L	G	L	N	N	L	L	L	N
2. Less hazard. & flam. storage	G	↑	↑	↑	↑	↑	↑	↑	L	N	↑	↑	↑	↑	N	↑
3. Wood working factories	↑	|	|	|	|	|	|	|	↑	↑	|	|	|	|	↑	|
4. Repair garages	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
5. Aircraft	G	S	L	N	S	L	L	L	L	N	N	N	L	L	N	N
F. GENERAL COMMERCIAL																
1a. Service stations & nonvital vehicle storage garages	L3	L3	N	N	L3	L	L	L	L	N	N	N	L	L	L	N
1b. Storage & parking of emergency vehicles	G	G	L	L	G	L	G	G	G	L	L	N	L	L	L	N
2a. Wholesale stores & general warehouses	L	G	L	L	G	L	L	L	L	N	N	N	L	L	L	N
2b. Retail stores, incl. drinking & dining under 100	G	G	↑	↑	G	↑	G	G	G	L	G	L	G	G	G	L
2c. Office bldgs. - low rise	G	G	|	|	G	|	↑	G	↑	L	L	N	↑	G	G	L
2d. Office bldgs. - high rise	S	S	↓	↓	S	↓	↓	G	↓	L	L	N	|	G	G	L
2e. Print shops, factories, industrial plants	G	↑	L	L	↑	L	G	L	G	L	L	N	|	L	L	N
2f. Police & fire stations, communication centers	S	↓	G	G	↓	G	S	S	S	G	G	L	↓	G	G	L
2g. Warehouse (emergency supplies storage)	S	S	N	N	S	L	G	G	G	L	L	L	G	G	G	L
3. Aircraft hangars, open parking garages	L3	G	L	L	G	L	L	L	N	N	N	N	L	L	L	N

TABLE C8-5 continued

OCCUPANCY	COMPONENTS															
	Building accessibility	Ext. non-struct. walls	Veneers - small scale	Canopies	Roofing units	Containerized and miscellaneous elements	Fire detection	Fire suppression	Life safety communication	Smoke removal	Stairs	Elevators	Vertical shafts	Horizontal exits	Public corridors	Private corridors
	1.1	2.1	2.2	2.3	2.4	2.5	3.1	3.2	3.3	3.4	4.1	4.2	4.3	5.1	5.2	5.3
G. SPECIAL FACILITIES																
1. Ice plants, factories & workshops using noncombustibles and nonexplosives	L	L5	L	L	L	L	L	G	L	L	G	L	L3	G	G	L
2. Lifeline facilities, utilities, power plants	G	L5	L	L	G	L	G	S	G	S	G	G	G	G	G	L
H. HOTELS & APARTMENT HOUSES																
1. Hotels, convents, & monasteries	G	S	G	G	G	G	S	S	S	S	S	S	S3	G	S	G
2. Apartments - low rise	G	G1	G1	G1	G	G	G	S	G	G	G	G6	S	G	G	L
3. Apartments - high rise	G	S	G	G	G	G	S	S	S	G	S	S	S	G	G	L
I. DWELLINGS																
1. Dwell's, lodg. houses, sheds	L	L	L	L	M	M	G	G	L	L	G	M	M	-	-	M
J. MISCELLANEOUS																
1. Private garages	L	M	M	M	M	M	L	G	L	L	L	M	M	-	-	-

GENERAL NOTES:

1. Occupancies occupying a minor portion of another building shall not have any component criteria of a lower rating than the basic building.
2. Where one component is supported by another, the supporting component must have a performance level equal to or greater than the supported component.
3. Where the collapse of one component can seriously damage an adjacent component, the component with the potential for collapse must have a performance level equal to or greater than the adjacent component.

TABLE C8-5 continued

OCCUPANCY	COMPONENTS															
	Full height area and separation partitions	Full height structural fireproofing	Full height other partitions	Partial height partitions	Ceilings - fire membrane	Ceilings - non-fire membrane	Equipment - ceiling mounted	Equipment - wall mounted	Equipment - free standing unstable	Equipment - free standing stable	Furniture - unstable	Furniture - stable	Art work - ceiling mounted	Art work - wall mounted	Art work - free standing, unstable	Art work - free standing, stable
	6.1	6.2	6.3	6.4	7.1	7.2	8.1	8.2	8.3	8.4	9.1	9.2	10.1	10.2	10.3	10.4
G. SPECIAL FACILITIES																
1. Ice plants, factories & workshops using noncombustibles and nonexplosives	L3	G	L	L	G	L	L	L	L	L	L	L	L	L	L	N
2. Lifeline facilities, utilities, power plants	G	G	G	G	G	G	S	S	S	S	G	G	G	G	G	G
H. HOTELS & APARTMENT HOUSES																
1. Hotels, convents, & monasteries	G	G2	G	G	S	G	G	G	G	L	G	L	G	G	G	L
2. Apartments - low rise	L	G	L	L	G	L4	G	G	G	L	L	N	L	L	L	N
3. Apartments - high rise	G1	S	L	L	S	L4	G	G	G	L	L	N	G	G	G	L
I. DWELLINGS																
1. Dwell's, lodg. houses, sheds	G	L	L	L	L	N	L	L	N	N	N	N	L	L	N	N
J. MISCELLANEOUS																
1. Private garages	L	L	N	N	L	N	N	N	N	N	N	N	N	N	N	N

PERFORMANCE NOTES:

1. May be reduced one level if properly landscaped.
2. May be reduced one level if properly landscaped and building is only one story.
3. Must be raised one level if building is more than three stories or 40 feet high.
4. Must be raised one level if not light weight; metal frames must stay attached to building.
5. Must be raised one level if building is in urban area; system must stay attached to building.
6. Elevator does not need to operate if building is less than 40 feet high.

REFERENCES

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State of California Senate. 1972. Bill 519 (March 8).

Timoshenko and Young. 1955. *Vibration Problems in Engineering*, 3rd Ed. Princeton, New Jersey: D. van Nostrand Co.

U.S. Department of Defense. 1974. *Seismic Design for Buildings*, Tri-Service Manual. Washington, D.C.: U.S. Government Printing Office.

Chapter 9 Commentary

WOOD

9.1 REFERENCE DOCUMENTS

Unlike some structural materials such as concrete or steel, wood construction practices have not been codified in a form that is standard throughout the country. Although heavy timber design practices generally follow the *National Design Specifications for Stress Grade Lumber and Its Fastenings* (22NDS), this document does not specify either simple or critical construction practices. There is a similarity of construction in lightweight wood framing throughout the country, but there is no single code of practice that is generally accepted. The closest approximation is probably Chapter 25 of the *Uniform Building Code*. Other reference documents are listed in Sec. 9.1.

It is not illogical to suggest that the framing practices specified in the *UBC* document be used throughout the country since wind design often governs over earthquake design even in highly seismic areas. The practices used for earthquake resistance are in large part those used to provide wind resistance.

The general provisions of Chapter 9 specify the construction requirements necessary to provide earthquake resistance although many are also related to gravity load resistance. Since these requirements are not covered in any comparable document except the *UBC*, they are included here for clarity and completeness.

9.2 STRENGTH OF MEMBERS AND CONNECTIONS

Since the loading provisions of Chapters 3 and 4 are based on a level of load resistance at yield point while normal code timber stresses must consider factors of safety, long-term deflection, etc., some adjustment must be made to tabulated stresses as given in the reference documents. This adjustment has been set at 200 percent of basic working stresses with the strength of members and connections subject to seismic forces acting alone or in combination with other prescribed loads being determined using the appropriate capacity reduction factors given in Sec. 9.2.

In the case of steel, the corresponding point has been averaged at about 1.7 times the tabulated working stress limitations. In the case of concrete, the adjustment is about 1.4. Capacity reduction factors also are specified for steel and concrete.

Wood has a variety of load factors and many of the accepted stresses do not have a constant relationship to an elastic limit or even an ultimate limit. When determining the factor for wood, consideration was given to the time effect of loading, the normal variability in strengths as related to both wood density and defects, and manufacture.

9.3 SEISMIC PERFORMANCE CATEGORIES A AND B

Buildings assigned to Categories A and B are required to meet minimum construction as required without consideration of seismic forces except for anchorage of concrete or masonry walls to floors and roofs as specified in Sec. 3.7.6.

In many parts of the United States where recent editions of the UBC are not used, minimum wall bracing is required for wood frame buildings three stories in height to prevent racking. These are similar to the Federal Housing Administration's (FHA) *Minimum Property Standards*. One common form of bracing has been omitted in these provisions: let-in 1 by 4 or 1 by 6 diagonal bracing members. The original tests for this type of bracing were reported by the U.S. Department of Agriculture's Forest Products Laboratory in 1929; however, in those tests the let-in bracing was combined with horizontal timber sheathing boards. The San Fernando earthquake demonstrated that the expected strength of walls with let-in bracing is greatly reduced when sheathing boards are not used.

9.4 SEISMIC PERFORMANCE CATEGORY C

Buildings assigned to Category C construction are required to meet requirements that are somewhat more restrictive than those for Categories A and B. Materials (e.g., screws, lag screws, fiberboard diaphragms, eccentric timber joints) and practices that have performed poorly in past earthquakes are regulated.

9.5 SEISMIC PERFORMANCE CATEGORY D

The additional requirements for buildings assigned to Category D correspond roughly to the requirements for ordinary construction in highly seismic areas of the United States. Only timber or plywood rated sheathing diaphragms are permitted and the other related materials are limited for bracing purposes to the top floor of a timber building.

The lack of adequate cyclic or dynamic test data on sheathing materials other than wood and plywood and the observed poor performance in past earthquakes of structures utilizing sheathing material for lateral resistance dictates that there be limits on the application of these materials. Recent dynamic testing of gypsum wall board sheathed panels has confirmed that the strength of this material degrades.

9.6 SEISMIC PERFORMANCE CATEGORY E

The requirements for buildings assigned to Category E further restrict the use of plaster, gypsum, particle board, wallboard, and fiberboard as bracing elements and require blocked diaphragms. These requirements apply only to those essential facilities in areas with the highest seismic exposure in the United States.

The greater need for reliable performance of structures in this category requires that panel materials be restricted to those for which there is adequate knowledge of behavior under dynamic loadings.

9.7 CONVENTIONAL LIGHT TIMBER CONSTRUCTION

Conventional light timber framing consists of light framing where sizes of studs, joists, and rafters are generally determined from tables and construction details are based on common practice possibly modified by local building codes or FHA *Minimum Property Standards*. These buildings are often sheathed with non-timber materials such as plaster, sheet rock, particle board, or other similar materials. Lateral resistance to wind or earthquake is usually not calculated but is determined by empirical rules such as are noted in Sec. 9.3.1 and 9.7.2.

9.8 ENGINEERED TIMBER CONSTRUCTION

Engineered construction includes timber framed buildings where loads and forces are calculated and the required resistance is provided according to the tested or designed capacity of the resisting elements. Special requirements (including those for torsion) are given for all types of shear panel construction including diagonal sheathing, plywood, and other materials.

REFERENCES

U.S. Department of Agriculture Forest Products Laboratory. 1929. *The Strength and Rigidity of Frame Walls*. Washington, D.C.: USDA.

Chapter 10 Commentary

STEEL

10.1 REFERENCE DOCUMENTS

The reference documents presented in this section are the current standard specifications for design of steel members and their connections in buildings as approved by the American Institute of Steel Construction (AISC), American Iron and Steel Institute (AISI), and the Steel Joist Institute (SJI).

10.2 STRENGTH OF MEMBERS AND CONNECTIONS

The modifications to standard specifications necessary to make them compatible with the design requirements of Chapter 3 and the force levels specified in Chapters 4 and 5 and those made to minimize potential brittle modes of failure are specified. Capacity reduction factors are provided so that in the future explicit determination of member strength factors can be expedited. The modifications only affect designs involving seismic loads.

The capacity reduction factor of 0.9 for members and connections was selected primarily to account for uncertainties in design and construction. Connections of members have generally been a critical element in failures during past earthquakes. Therefore, a capacity reduction factor of 0.67 was introduced to increase the capacity of those connections that do not develop the full strength of the member. A ϕ factor of 0.8 was selected for partial penetration welds subjected to tension stresses because there has been little experience with this type of connection in past earthquakes.

It frequently has been found that optimum performance is obtained if connections fully develop the minimum capacity of the members of the seismic resisting system framing into a joint. Somewhat brittle-type failures have been observed when the capacity of connections are reached before that of the member. In order to provide a greater than usual margin of safety on braced frame connections, SEAOC (1974) recommends that connections be sized without consideration of the one-third increase in allowable stress usually permitted unless the member capacity is fully developed. This concept is extended to moment frames by providing the same conservatism for moment frame connections as for braced frame systems.

It has been demonstrated by tests that a moment connection composed of welded flanges with a bolted web connection designed to carry the shear can develop the plastic capacity of steel sections (Huang et al.,1971 and 1973; Regec et al., 1972; Rentschler and Chen, 1973, 1974, 1975, and 1976; Parfitt and Chen, 1974; Popov and Stephen, 1970.)

When designing the connection to fully develop the member, the strengths of the connecting parts are determined using the factor increasing the allowable stress that is given in Sec. 10.2.1. This may create a step function in determining strengths. However, in design, a decision is made initially on whether or not the member strength will be developed so that the step should not create a design problem.

10.2.1 Structural Steel

Modifications are given for Ref. 10.1 (AISC specifications).

10.2.1.1 Load Combination

The load effects determined from the load combinations specified in Sec. 3.7.1 are required to be equal to or less than the actual strengths of members and connections. The allowable stress levels specified in Part 1 of Ref. 10.1 do not identify this condition and are not applicable. It is assumed, unless specifically described otherwise, that the strengths are linear, elastic allowable stresses modified to meet the elastic limit of the structure. The design for the combination of dead and live loads and impact, if any, is not modified from the current specifications. Information leading to the determination of member and connection strengths was being developed but was not yet available when these provisions were originally drafted. The LRFD Design Specification was published in 1986 and is referenced in the 1988 Edition of the *Provisions*. Material in the "Appendix to Chapter 10" provides the means whereby the LRFD specification (Ref. 10.6) can be used to modify the provisions of Chapter 10. For use of the elastic design specification, a modifier of 1.7 and a capacity reduction factor of $\phi = 0.9$ on working stress values were chosen after a review of a number of items such as:

- The margin of safety between the yield strength and allowable stress of short columns.
- The margin of safety between the yield strength and allowable tensile stress.

- The margin of safety of compression members, which varies between 1.7 and 1.9 (Ref. 10.1; Johnston, 1976).
- The increase permitted on connecting devices in Part II of Ref. 10.1, which is 1.7 (Ref. 10.1). The actual margin of safety is often higher (Fisher and Struik, 1974; Galambos and Ravindra, 1976).

10.2.1.2 Euler Stress

Since the level of design is the same as contemplated in the definition of P_e on Page 5-60 of Ref. 10.1, the 12/23 modifier of F'_e is removed.

10.2.1.3 Member Strength

Proportioning members of seismic resisting braced frame systems of a building that has been designed by plastic analysis for gravity loads should be based on the strength of members as specified in Part 2 of Ref. 10.1. However, the analysis should be based on the elastic analysis described in Sec. 3.1 of the *Provisions*. Thus, the current references to plastic analysis methods and the load factors are not used.

10.2.1.4 Shear Strength

The allowable shear stress specified in Sec. 1.5.1.2 of Ref. 10.1 is 0.40 F_y. When multiplied by 1.7, the value becomes 0.68 F_y. This is higher than the 0.55 F_y given in Sec. 2.5 of Ref. 10.1 and the 0.60 F_y given in Sec. 10.2.1.4. This difference is discussed in the commentary of Ref. 10.1. The value given in Ref. 10.6 is 0.60 F_y.

10.2.1.5 P-delta Effects

This section provides modification to the interaction equations when the P-delta effects are explicitly determined in conformance with Sec. 4.6.2. In columns, the reductions given to the allowable stresses are in part a result of the consideration of member P-delta effects. These P-delta reductions are modified in Ref. 10.1 by a K factor that is a recognition of the effect of end restraint in the member P-delta relationship. In beam-columns, the P-delta effect also is considered as an increase (or decrease) to the moments at the end of the columns expressed as a function of (Ref. 10.1; Johnston, 1976; Galambos, 1968):

$$C_m/1 = f_a/F'_e.$$

The bases for the values of this ratio in braced systems are well documented. The selection of the value of C_m in unbraced frames was an

The bases for the values of this ratio in braced systems are well documented. The selection of the value of C_m in unbraced frames was an approximation applicable primarily to designs where significant applied horizontal forces are not present. Since the advent of computer analyses, the solution of the secondary effects resulting from deflection has become much easier. In most cases, with significant horizontal force displacements (but limited by drift requirements) the first iteration of deflection is sufficient. It is possible that some members, such as weak axis columns depending on end support conditions, may have critical stress occur at the midstory rather than the column ends. Thus, the stress limits specified for braced frames should not be exceeded.

10.2.2 Cold Formed Steel

The allowable stress levels for cold formed and stainless steels of Ref. 10.2 and 10.3 are not applicable to the force levels in the earthquake analysis specified in Chapter 3. As an interim measure the strengths of the members governed by these provisions are determined using basic stresses increased by 1.7 and using $\phi = 0.9$.

Three approaches for determining the strength of steel deck diaphragms have been included. This was done to clarify the use of the steel deck diaphragm ϕ factor in the strength method of this chapter.

10.2.3 Steel Cables

The allowable stress levels of steel cable structures specified in Ref. 10.6 are modified for seismic load effects. The value of 1.5 T_4 was chosen as a reasonable value to compare with increases given to other working stress levels.

10.3 SEISMIC PERFORMANCE CATEGORIES A AND B

No special requirements for seismic design of buildings assigned to Category A or B were deemed necessary.

10.4 SEISMIC PERFORMANCE CATEGORY C

Detail requirements for buildings assigned to Category C are given.

10.4.1 Space Frames

Where moment resisting frame systems are used for the seismic resisting system, they must be Ordinary Moment Frames as a minimum. Ordinary

Moment Frames are assumed to respond to the design earthquake by requiring a limited amount of nonlinear behavior. For this type of moment frame, proportioning of members and their connections is based on the requirements of the referenced specifications as modified by Sec. 10.2 for making working stress values compatible with seismic design. For these types of frames no change is provided to local buckling criteria in Appendix C of Ref. 10.1 and in Ref. 10.2 and 10.3.

10.4.2 Braced Frames

Braced frames must conform to Ref. 10.1, to Ref. 10.2, or to Ref. 10.3. The connection requirement assures a minimum of ductility. The requirement for equivalent strengths in opposing directions protects against progressive inelastic deflections accumulating in one direction that could lead to stability problems.

10.5 SEISMIC PERFORMANCE CATEGORY D

10.5.1 Moment Frames

Where a moment resisting frame system is used as the seismic resisting system it must be a Special Moment Frame as specified in Sec. 10.7. An exception is made permitting Ordinary Moment Frames for one- and two-story buildings. This exception is based on the generally good experience record of such buildings during earthquakes.

Minor structures and structures with light metal or wood cladding designed without special requirements for nonlinear ductile behavior have performed well even during strong earthquakes. However, major structures in areas of high seismicity and those minor structures housing emergency occupancies should be provided with the full provisions for inelastic performance specified by Sec. 10.7. A major structure in this instance is defined as a building over two stories. It is conceivable that some one- and two-story structures should be considered major structures and that some buildings of four or five stories, particularly those with light flexible cladding, should not be classified as major structures. Some judgment and leniency should be exercised in enforcing the two-story limitation.

10.5.2 Braced Frames

Special details are required for either concentrically or eccentrically braced frames. The details are specified in Sec. 10.8 and 10.9.

10.6 SEISMIC PERFORMANCE CATEGORY E

10.6.1 Moment Frames

Where a moment resisting frame system is used as the seismic resisting system, it must be a special moment frame as specified in Sec. 10.7. An exception is permitted for one-story buildings for which ordinary moment frames may be used.

10.6.2 Braced Frames

Concentric braced frame systems may only be used as part of a dual system. Special details are required for either concentrically or eccentrically braced frames. The details are specified in Sec. 10.8 and 10.9.

10.7 SPECIAL MOMENT FRAME REQUIREMENTS

Structures having Special Moment Frames designed to meet the requirements of Sec. 10.7 are intended to have the capability of significant nonlinear deformation. The sizing of members is based on the limit of an elastic model as specified in Sec. 3.1. The nonlinear capability is provided by meeting the special requirements in this section.

10.7.1

The statement regarding M_p is added to the specifications so that it can be used to define the flexural strength of a frame member. This definition of strength is obviously not the elastic limit of the member but, as a consequence of strain hardening, it is felt to be a reasonable limit to represent the point at which the frame as a whole will start to substantially deviate from linear response. The fact that the mean yield strength of the material is in excess of the minimum specified yield strength also supports this design concept.

10.7.2

For this type of moment frame the steels to be used are limited to those whose properties are similar to the steels used in tests to demonstrate the nonlinear behavior of structural members and joints (Lehigh University, 1967-1976; Popov and Stephen, 1970; Popov et al., 1975; Bertero et al., 1973; Krawinkler et al., 1971; Becker, 1971). Other steels exhibiting similar ductility and strain hardening characteristics such as those listed also would be appropriate.

10.7.3

Sec. 2.3.1 of Ref. 10.1 is deleted as not applicable to unbraced frames. The maximum axial load on columns of 0.6 P_y for Special Moment Frames is provided to reflect the recommendations from recent tests. The upper limit for the axial forces is lowered from 0.75 P_y, as specified in Sec. 2.3.2 of Ref. 10.1, to 0.6 P_y for two reasons:

- The uncertainties involved in predicting the maximum axial forces that can be induced during a severe earthquake are so great that it is convenient to be more conservative than in case of design for standard loadings.
- Columns in a moment resisting frame system (ductile or nonductile) excited by severe earthquake ground motion can be subjected to cycles of inelastic moment reversals.

Test results (Popov et al., 1975) have shown that when a column is under a constant axial force $p \geq 0.6\ P_y$ and is subject to reversals of moments inducing yielding, local buckling develops in the columns during first reversal of inelastic moment and, when this occurs, the axial force cannot be maintained.

10.7.4

The actual location of points of inflection in columns when the frame is deforming nonlinearly is not known. Thus, the shear and moment requirements at a column splice are difficult to accurately access. The use of partial penetration welds for column splices produces a point that could result in a brittle-type frame failure if the level of stress is critical at any time during the response of the frame. In order to provide a conservative guide to the determination of when partial penetration welds can be used, the following criteria are provided by the provisions:

1. A conservative estimate of joint moment capacities is required assuming the yield of the critical sections at the joint are 125 percent of the minimum specified yield strength;

2. The potential movement of the point of inflection within the column height is determined by assuming that one column joint is stressed to one-half of its plastic capacity and the other joint is stressed to its full plastic capacity; and

3. The effect of vertical acceleration is considered by using the load combinations of Sec. 3.7.1. In some cases columns do not have a point of inflection within a story height. For these cases it could be unconservative to design the splice to comply only with cases 1 and 2 above. Thus, it is emphasized

that the load effects resulting from the loads specified in Sec. 3.7.1 also should be considered.

10.7.5

In addition to the shear stresses resulting from the elastic analysis of the system under the specified loads, shear stresses should be determined based on the assumption that the full flexural strengths of the elements are reached through nonlinear displacement of the frame members. The critical sections may be either in beams or in columns. Frequently this may be only a nominal change in the shear design requirements. It is felt that the shear requirements should be consistent with the actual response of the frame to the design earthquake. If the members are oversized, the actual inelastic displacement of the frame will not be the same as assumed when assigning the load modifiers in Sec. 3.7.1. The resulting increase in the design shear can be significant.

Research has been performed on beam-column joint panel zones and methods have been proposed for determining the panel zone shear capacity with and without shear reinforcement (Becker, 1971; Bertero et al., 1973; Krawinkler et al., 1971). Panel zone shears frequently have been determined assuming the joint moments equal to the sum of the beam (or columns) moment capacities on each side of the joint. This is a simple and conservative method of determining panel zone shears but usually results in excessive reinforcement requirements. However, it usually is not possible to develop this joint moment on the frame before total frame instability occurs. Also formation of hinging by shear in restricted areas may provide stable nonlinear response. In most cases, the provisions of Sec. 10.7 permit reduction in the amount of reinforcement required when an approximate frame analysis is made with deflections twice those determined using the prescribed forces. The factor of 2 is arbitrary but would provide elastic panel zone response well beyond the deformations represented by the design forces at the elastic limit of the structure.

10.7.6

Connections usually should be designed to develop the joint capacity rather than the connection stresses resulting from the effects of the specified earthquake loading. This is to ensure that ductile behavior will occur in the members. Connections could be devised, however, to be capable of providing adequate nonlinear response in themselves. This should be demonstrated by proper analyses or tests.

10.7.7

Sec. 2.9 of Ref. 10.1 is modified to delete reference to plastic design procedures for design of the seismic resisting system so as to be in conformance with the requirements for an elastic analysis as specified in Sec. 3.1.

10.8 CONCENTRICALLY BRACED FRAME REQUIREMENTS

Braced frames are divided into two basic types, concentrically and eccentrically braced frames. Eccentrically braced frames are described in Sec. 10.9. The provisions of Sec. 10.8 apply to concentrically braced frames in Seismic Performance Categories D and E.

Ductile energy dissipation by means of reversible inelastic lateral distortions is the basic justification for seismic design based on design forces factored down from those expected in a structure that responds in a fully elastic manner during a strong earthquake. Braced frames have a limited capacity for reversible inelastic distortions. Inelastic axial stretching and shortening of slender members results in a residual elongation of the members. Inelastic axial stretching and shortening of more stocky members is reversible up until significant compression buckling takes place. Tests (Popov and Black, 1961, and Jain, Goel, and Hanson, 1980) show that after buckling, an axially loaded member loses compression capacity with repeated inelastic load cycles and does not return to its original straightline position.

It is for these reasons that braced frames are allowed for tall or special buildings in the higher Seismic Performance Categories only as part of a dual system. In addition to this limitation, the basic design force is higher than that of a ductile moment frame system due to the difference in the R factors.

Bracing members are occasionally used within moment resisting frame systems to control drift. The design of the moment resisting frame system must consider the presence of the bracing as well as its potential failure as required by Sec. 3.3.4. In addition, the bracing and the members connected to the bracing must be designed to satisfy the provisions of this section.

When an axially loaded brace buckles in compression, bowing out of line, several things can happen:

- The load capacity of the buckling brace drops, causing the tension acting brace to take more load--if there is resistance to the tension.

- The bowing of the member may rotate the ends of the brace excessively at the connection causing local failure.

- The bowing causes local buckling failure near mid-span of the brace.

- Out-of-plane bowing of the brace can cause nonstructural failure in the brace encasement. A difference of 20 ksi between opposing compression acting and tension acting braces 25 feet long causes a bow of about 3 inches.

- The building displacements become one-sided without balanced bracing.

- The building becomes less stiff due to the elongation of the brace through tension yielding.

It is obvious that in tension rod "X" braced industrial buildings without rigid walls only Item 1 of the effects of brace bowing is of serious consequence. However, for commercial buildings, all of the above effects are serious. Low L/r ratios and low stresses will reduce or prevent brace bowing. But rotation of rigid brace end connections and local buckling fracture of brace elements (tube walls and stitched angles) need to consider the latest test information--which was not debated enough by the BSSC committee working on these provisions for inclusion in the 1988 Edition. Designers should consider the latest physical test results. Analytical test results need to be considered with caution because of the modeling assumed.

A bracing system with brace members connecting to columns between floor levels is undesirable for seismic resistance. Deformations of the bracing system due to tensile yield and/or buckling may cause lateral deformations of the connected column sufficient to cause column buckling and subsequent collapse.

10.8.1 Bracing Members

In the post-buckling range, the compressive strength of axially loaded members deteriorates and the hysteresis loops take on a severely pinched shape (see Figure C10-1). Braces with small L/r will dissipate more energy because in the post-buckling range they will undergo cyclic inelastic bending that slender braces will not. Very slender braces have almost no stiffness while they straighten out from a buckled configuration and in the straightened configuration (tension) pick up stiffness very rapidly.

Unfortunately, the choice of a brace with a small L/r is not always ideal either. Such braces undergo cyclic inelastic bending in the post-buckling range. The curvatures associated with this cyclic inelastic bending may be very large and local buckling has to be expected. This local buckling may be severe and may lead to localized kinking of plate elements that, in turn, may cause crack propagation

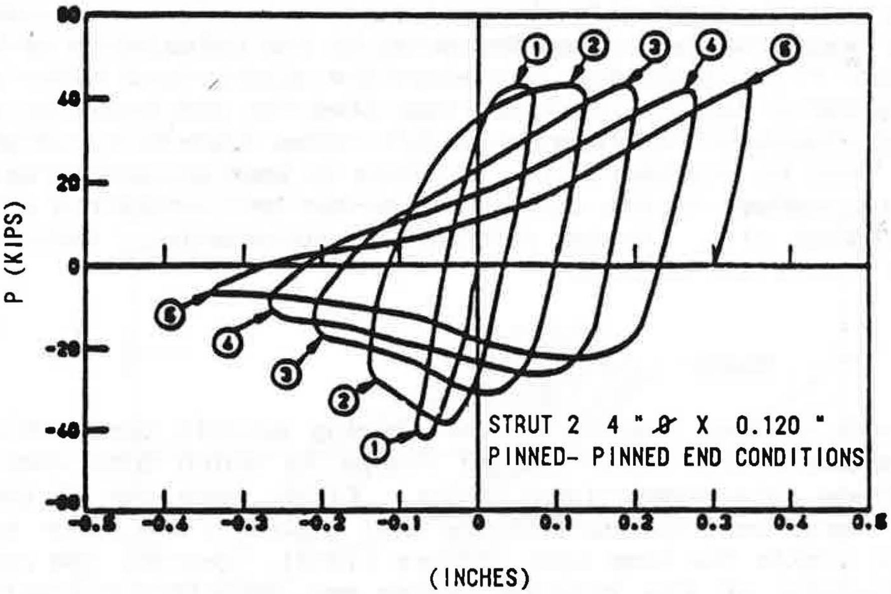

FIGURE C10-1
Cyclic response of an axially loaded element
(Zayas, Popov, and Mahin, 1980).

and fracture. Such fracture has been observed rather early in tests of tubular bracing members. It is noted that this characteristic is likely more prevalent in square tube braces. Thus, the new limit of $90/\sqrt{F_y}$ has been imposed for walls of rectangular tubes. Consideration should be given to filling tubes with concrete or using other means to inhibit local buckling.

The purpose of the lateral force distribution provision is to require nearly equal strength bracing in opposed directions on every line of bracing (or at least on closely adjacent lines). This is to prevent an accumulation of inelastic deformations in one direction only (the "ratcheting" effect).

Since Appendix C of Ref. 10.1 provides for reduction of allowable stresses for elements that exceed the permitted width-thickness ratios, it clearly should not be applied to elements of members that are expected to exceed allowable stresses. Thus, the limits of Part 2 of Ref. 10.1 are applied.

The integrity of a brace in the post-buckling range requires that buckling of individual segments between stitches be prevented. Considering that the buckled brace may take on a significant curvature and that end restraints are often neglected in the calculation of KL/r for the brace, it is necessary to reduce the slenderness ratio of individual segments to a value lower than that for the brace as a whole. Note that the reference here is to L/r rather than KL/r. Rigid shear transfer must be provided across stitches so that the shears associated with the curvature in the buckled brace can be transferred across the stitch without slip. Welded stitches are recommended. Machine bolted stitches should not be used.

10.8.2 Beams

Frames with chevron bracing or vee bracing exhibit two problems that set them apart from other braced frames in which both ends of the braces frame into beam-column joints. First, once one of the braces buckles, large vertical deflections will occur in the floor system at the joint within the beam span (Figure C10-2). Second, the post-buckling strength of the bracing system may deteriorate significantly because the tensile force that can be developed in the tension brace can exceed the decreasing post-buckling strength of the compression brace only by the additional force necessary to cause hinging of the floor beam. This force will be nil unless the beam is at least continuous from column to column. This additional force can be made larger by increasing the size of the beam and by making the beam-to-column connection a moment resisting connection.

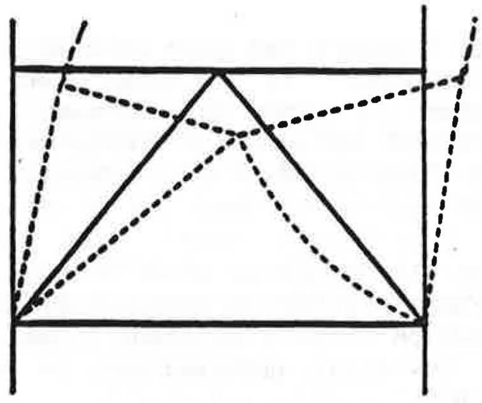

FIGURE C10-2
Chevron bracing in post-buckling stage.

Once a brace in a chevron or vee system buckles, it will not straighten out fully under load reversal, the opposite brace will buckle at load reversal, and the story shear resistance of the bracing system will be governed by the post-buckling behavior of the two opposed braces. The consequence will be a deteriorating story shear-story drift relationship like that shown in Figure C10-3. The only reason why the relationship in Figure C10-3 does not exhibit more rapid deterioration is that in the test structure, from which this relationship was obtained, the chevron bracing system was surrounded by a ductile moment resisting frame that provided a considerable portion of the shear resistance once the bracing had buckled.

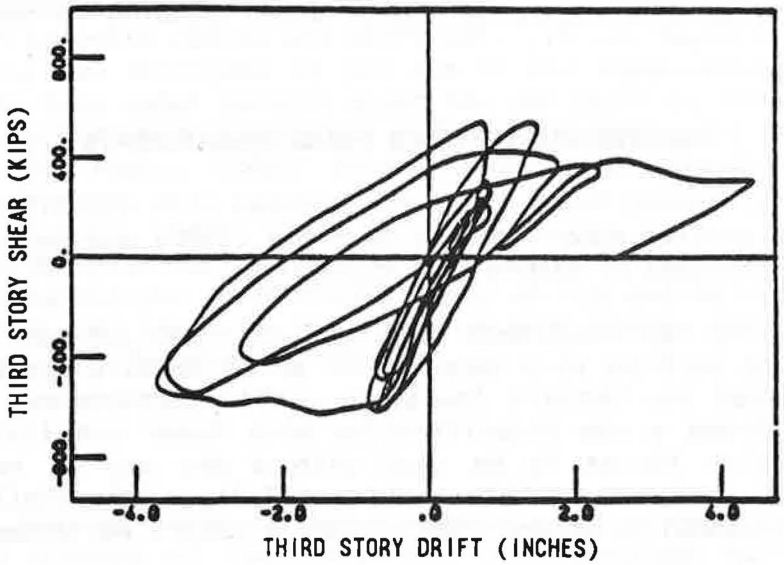

FIGURE C10-3
Story shear-story drift diagram for a frame structure with chevron bracing (Wallace and Krawinkler, 1985).

10.8.3 Columns

It is important to prevent unanticipated yielding or buckling from occurring in columns of braced frames in order to prevent serious gross stability problems for the structure as a whole. Thus, the design force for columns must be a realistic estimate of the maximum forces to which they might be exposed. The real capacities of bracing members, not the design force, must be used.

10.8.4 Bracing Member Connections

In a severe earthquake, the force demand on a brace in tension is expected to exceed the brace tensile yield strength. In order to avoid brittle failure at the brace connections, the connections must be designed to develop the tensile yield strength of the brace. These same minimum strength requirements also apply to the beam connections that are part of the bracing system.

For bolted connections, the provisions of the LRFD specification are followed for the net area requirement.

The requirement for clear end distance in a gusset plate connection is intended to permit rotation of the end of the bracing member due to out-of-plane buckling without generating large and damaging stresses in the gusset plate. The point on the gusset to which clear end distance is measured is not the end of the gusset but a line connecting the points at which resistance to out-of-plane bending of the gusset is restrained.

10.9 ECCENTRICALLY BRACED FRAME REQUIREMENTS

The eccentric braced frame (EBF) is credited with ductility similar to that of the special moment resisting frame (SMRF) and has many special provisions intended to ensure that ductility.

Recent research has shown that this type of frame can have the ductile properties to perform in a manner similar to ductile moment frames if it is detailed for ductile inelastic cyclic performance. Chapter 3 gives this frame a new classification with fewer use limitations and reduced design forces to be used with a new set of very detailed requirements included in this section (10.9). The detailed requirements are intended to ensure good inelastic cyclic performance.

An EBF is a braced frame with at least one end of each brace intersecting a beam, eccentric to the beam intersection with the column or the opposing brace. The section of the beam between opposing braces, or between a brace and beam-column intersection, is called the "link beam" and is the element of the frame counted on to provide inelastic cyclic yielding.

The design intent of the eccentric braced frame, in contrast to that of the concentric braced frame, is to provide a ductile link where the structural system will yield in lieu of buckling of its braces when such frame experiences dynamic loads in excess of its elastic strength. The integrity of this system depends on the requirement that the braces have a design strength that is at least 50 percent greater than the strength at which the link beam begins to yield. This precludes potential buckling of the braces and promotes a controlled inelastic deformation that is capable of significant energy absorption.

The rotation demand on the link beam is a multiple of the lateral rotation (or drift) of the frame as a whole, a multiple (rotation factor) that is related to the geometry of the frame. Since most link beams are short enough to cause shear yielding with large rotation factors, most of the EBF research at the University of California at Berkeley (UCB) has tested the cyclic inelastic capacity of the link beams with shear yielding at large rotations. Most of the provisions in Sec. 10.9 are concerned with limiting the link beam shear yield rotation to less than the test rotations and requiring details that tests indicate are necessary to ensure ductile rotation capacity.

It has been known from many moment frame panel zone tests that shear yielding is very ductile and that the inelastic shear force capacity based only on beam web shear area greatly underestimates the web-plus-flange inelastic shear capacity. However, the link beam shear yield rotation demands are much greater than panel zone demands, and the panel width to depth ratios are different. It was found that where the ratio of the web thickness to the sum of the width plus depth dimensions of the beam panel exceeds about 90, web buckling reduces the beam inelastic cyclic shear capacity. Therefore, based on tests at UCB (Hjelmstad and Popov, 1983), several provisions specify requirements for web stiffeners.

Most of the detailed EBF provisions have been developed by SEAOC, based on the recommendations of Professor Popov at UCB and on his interpretations of the EBF research conducted at UCB (Kasai and Popov, 1986; Popov, Kasai, and Engelhardt, 1987).

10.9.1

Link beams are the fuses of the eccentric brace structural system that are to be placed at locations that will preclude buckling of the braces. A link beam must be located at least at one end of each brace.

Compact sections meeting the more restrictive flange width to thickness ratio of $52/\sqrt{F_y}$ are required for the beam portions of eccentric braced frames in order to provide the beams with stable inelastic deformation characteristics. The same requirement is used for the beams of special moment resisting space frames.

10.9.2

The prohibition of link beam web doubler plates was required because tests (Becker, 1971, and Popov, 1980) have shown that they are not fully effective. Performance of eccentric braced frames relies heavily on the predictability of the strength and strain features of the link beam. It is considered not advisable to complicate the behavior of the link beam by allowing holes within it.

10.9.3

A link beam is considered to be balanced between shear yielding or moment yielding when the link beam length equals 2.0 M_s/V_s. A link beam of 1.6 M_s/V_s or less in length is considered to be a short link whose post-elastic deformation is controlled by shear yielding. Conversely, a link beam whose length exceeds 2.6 M_s/V_s is considered to be controlled by flexural yielding.

The ductility of an EBF is controlled by the inelastic rotation of the link segment relative to the rest of the beam. Tests (Roeder and Popov, 1978) have shown that when controlled by shear distortion, link beams with rotations of 0.10 radians performed adequately.

The link beam rotation limit for short links was set at 0.08 radians. At this level of short link rotation, floor damage is believed to be small.

The use of long links with lengths greater than 2.6 M_s/V_s has not been tested directly in EBF configurations. The rotational limit was conservatively set at 0.02 radians for long link beams. This is similar to the limit used for moment frames when one considers the drift limits of Sec. 3.8.

Limit check rotations are measured at the braced end of the link beam. The angle measured is located between the extension of the line passing through the center of each end of the link beam and the line passing through the center of each end of the brace beam portion. The displacement of the beam and center points may be obtained by multiplying the elastic deflections determined from the prescribed seismic forces by C_d. This is, of course, slightly conservative since the elastic curvature of the beam segments between hinges is ignored.

10.9.5

This requirement addresses the concern for the effect that substantial axial loads in the link beam could have on the link beam's inelastic deflection performance. It presumes that the web's capacity is fully utilized in shear and that reliance should be placed only on the flanges for axial and flexural capacity. Shear links with a length less than 2.2 M_s/V_s are considered to be controlled by shear.

Substantial axial loads occur when the link beam is required to transmit horizontal forces to or from the braces. Link beams have not been tested for this condition. It is recommended that this condition be avoided by careful placement of the link beams so that the link beams are not required to transmit the horizontal force component of braces or drag struts. The f_a should correspond to the force required to yield the link beam in shear unless it causes the flange to yield first.

10.9.6

Web stiffeners have been found by cyclic testing to be necessary at the ends of link beams and, in the case of shear links, at intermediate locations along the link.

For shear links, the stiffeners prevent buckling of the web. This assures development of the shear strength of the web and, additionally, reduces the cyclic deterioration of the web that can result from repetition of the buckling deformations.

For links that may have significant bending yielding (length between 1.6 and 2.6 M_s/V_s), an additional stiffener at the distance b_f from each end of the link is required to preclude the possibility of flange buckling.

10.9.10

Link beams adjacent to the column place large shear demands on the beam-to-column connection. Testing has shown the need to make these connections as strong in shear as possible by providing full penetration groove welding of the link beam flanges as well as by providing welded connections of the web for shear transfer.

10.9.12

The ratio of axial compressive strength of the brace to the initial yield strength of the link beam was selected at 1.5 because the upper strength limit of the link beam at this ratio approaches the upper strength limit of the brace in compression. The shear strength of the link beam is generally greater than the assumed strength based on 0.55 F_yd_t and can be more than 0.90 F_yd_t with close stiffener spacing. The 1.5 ratio is considered the minimum that will maintain the integrity of the brace. Conservatism is recommended for sizing the braces. This is particularly true for tube sections with thin wall thickness relative to their width.

10.9.13

The prohibition of the extension of the brace-to-brace connection into the link beam is intended to prevent physical attachments that might alter the strength and deflection characteristics of the link beam. It is not intended to prevent the center line intersection of brace and link beam from intersecting within the link. This intersection location is to be encouraged for it results in a greater moment in the link beam portion than in the brace portion of the beam.

10.9.14

It must be recognized that the requirement given does not protect the bottom level column from hinging. When a fixed-base detail is used, such hinging is inevitable if the drift of the frame becomes large enough. It is recommended that base level columns be compact sections and that the base connection be detailed to recognize and accommodate the potential hinging without failure of the column or of the brace-to-column connection.

APPENDIX TO CHAPTER 10

An appendix has been included to permit the design of structures by using the new LRFD Design Specifications, Ref. 10.6. The provisions included are tentative and should be used with caution for Categories D and E.

The LRFD methodology provides for more realistic strength models compared with allowable stress design. Chapter 10 currently uses a multiplier (factor of safety) of 1.7 times the allowable forces and moments to determine the strength of structural members and connections. This is satisfactory for structural members with limit states based on yielding. However, for limit states based on fracture, members have a factor of safety of 2. Chapter 10 specifies a resistance (capacity reduction) factor of $\theta = 0.9$. For limit states based on fracture, the actual resistance factor is $\theta = (1.7/2)0.9 = 0.765$. Because LRFD uses $\theta = 0.75$, the latter will give conservative results.

Chapter 10 gives special θ factors for connections that do not develop the strength of the members and for partial penetration welds in columns when subjected to tension stresses. The factors are 0.67 and 0.80, respectively. The corresponding values for LRFD are $(0.67/0.9)0.765 = 0.57$ and $(0.8/0.9)0.765 = 0.67$. Consequently, the following values of ϕ should be used when using the "Appendix to Chapter 10":

Connections that do not develop the strength of the member or structural system, including connection of base plates and anchor bolts, or do not conform to Sec. A10.3.6	$\phi = 0.57$
Partial penetration welds in columns when subjected to tension stresses	$\phi = 0.67$

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Chapter 11 Commentary

REINFORCED CONCRETE

11.1 REFERENCE DOCUMENT

The main concern of Chapter 11 is the proper detailing of reinforced concrete construction for earthquake resistance. The bulk of the detailing requirements in this chapter are contained in Appendix A of Ref. 11.1, *Building Code Requirements for Reinforced Concrete*, ACI 318-83. The 1983 seismic appendix to ACI 318 grew out of the Applied Technology Council's 1978 report, *Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC 3-06) and the review of that document, which resulted in amendments. The commentary for ACI 318-83 contains a valuable discussion of the rationale behind the seismic detailing requirements that is not repeated here.

11.1.1 Modifications to Ref. 11.1

The modifications noted for ACI 318-83 are of three general types:

- Changes in load factors necessary to coordinate the equivalent yield basis of this document;
- Changes that incorporate certain features of the seismic detailing requirements for reinforced concrete that have been adopted into the 1988 Edition of the *Uniform Building Code*; and
- Changes that coordinate with certain modifications that are being made from the 1983 to the 1989 Edition of ACI 318, Appendix A.

New in the 1988 Edition is a statement on prefabricated concrete buildings (Sec. 11.1.1.3).

The complexity of prefabricated concrete buildings and the multitude of structural systems, configurations, and details encountered in practice requires integration of knowledge obtained by: (1) selecting functional and compatible details for connections and members that are reliable and can be built with acceptable tolerances; (2) verifying experimentally the inelastic force-deformation relationships for welded, bolted, or grouted connections proposed for the seismic resisting elements of the building; and (3) analyzing the building using those connection relationships and the inelastic reversed cyclic loading effects imposed by the anticipated earthquake ground motions.

See the following references for guidance: A. Aswad, 1979, "Selected Precast Connections: Low-Cycle Behavior and Strength," in *Proceedings of the 2nd U.S. National Conference on Earthquake Engineering*, August 22-24, Stanford University; Applied Technology Council, 1981, *Proceedings of a Workshop on Design of Prefabricated Concrete Buildings for Earthquake Loads*, Report ATC-8; R. E. Englekirk, 1987, "Concepts for the Development of Earthquake Resistant Ductile Frames of Precast Concrete," *PCI Journal* (January-February); N. M. Hawkins and R. E. Englekirk, 1987, "U.S.-Japan Seminar on P/C Concrete Construction in Seismic Zones," *PCI Journal* (March-April); JSPS/NSF, 1986, *Proceedings of the Seminar on Precast Construction in Seismic Zones*, Vol. 1; D. S. Neille, 1977, Behavior of Headed Stud Connections for Precast Concrete Panels Under Monotonic and Cycled Shear Loading, thesis submitted in partial fulfillment of the requirements of Doctor of Philosophy, University of British Columbia; Prestressed Concrete Institute, 1983, *Proceedings of a Symposium on Planning and Design of a Precast Concrete Wall Building*; J. Stanton and C. Dolan, 1986, *Moment-Resisting and Simple Connections*, Research Report 1/4, Prestressed Concrete Institute.

11.2 STRENGTH OF MEMBERS AND CONNECTIONS

The strength reduction factors listed in Ref. 11.1 and the remainder of Chapter 11 are intended to define section or element strength.

The allowable loads on anchor bolts have been chosen to suit the capacity reduction factors in this document.

11.3 ORDINARY MOMENT FRAMES

Since Ordinary Frames are permitted only in Categories A and B, they are not required to meet any particular seismic requirements. Attention should be paid to the often overlooked requirement for joint reinforcement in Sec. 11.12.1 of Ref. 11.1.

11.4-11.5 INTERMEDIATE AND SPECIAL MOMENT FRAMES

The concept of Moment Frames for various levels of hazard zones and of performance is changed somewhat from the provisions of Ref. 11.1. Two sets of moment frame detailing requirements are defined in Ref. 11.1, one for "regions of high seismic risk" and the other for "regions of moderate seismic risk." For the purposes of this document, the "regions" are made equivalent to Seismic Performance Categories in which "high risk" means Categories D and E and "moderate risk" means Category C. This document labels these two frames the "Special Moment Frame" and the "Intermediate Moment Frame," respectively.

The level of inelastic energy absorption of the two frames is not the same. These provisions introduce the concept that the R factors for these two frames should not be the same. Use of Ref. 11.1 with seismic provisions currently in model building codes would imply that the equivalent R factors were indeed the same. The preliminary version of these provisions (ATC 3-06) assigned the R for Ordinary Frames to what is now called the Intermediate Frame. In spite of the fact that the R factor for the Intermediate Frame is less than the R factor for the Special Frame, use of the Intermediate Frame is not permitted in the higher Performance Categories (D and E). On the other hand, this arrangement of the provisions encourages consideration of the more stringent detailing practices for the Special Frame in Category C because the reward for use of the higher R factor can be weighed against the higher cost of the detailing requirements. These provisions also introduce the concept that an Intermediate Frame may be a part of a Dual System in Category C.

The differences in the performance basis of the requirements for the two types of frames might be briefly summarized as follows (see the commentary of Ref. 11.1 for a fuller discussion of the requirement for the Special Frame):

- The shear strength of beams and columns shall not be less than that required when the member has yielded at each end in flexure. For the Special Frame, strain hardening and other factors are considered by raising the effective tensile strength of the bars to 125 percent of specified yield. For the Intermediate Frame, an escape clause is provided in that the calculated shear using double the prescribed seismic force may be substituted. Both types require the same minimum amount and maximum spacing of transverse reinforcement throughout the member.

- The shear strength of joints is limited and special provisions for anchoring bars in joints exist for Special Moment Frames but not Intermediate Frames. Both frames require transverse reinforcement in joints although less is required for the Intermediate Frame.

- Closely spaced transverse reinforcement is required in regions of potential hinging (typically the ends of beams and columns) to control lateral buckling of longitudinal bars after the cover has spalled. The spacing limit is slightly more stringent for columns in the Special Frame.

- The amount of transverse reinforcement in regions of hinging for Special Frames is empirically tied to the concept of providing enough confinement of the concrete core to preserve a ductile response. These amounts are not required in the Intermediate Frame and, in fact, stirrups in lieu of hoops may be used in beams.

- The Special Frame must follow the strong column/weak beam rule. Although this is not required for the Intermediate Frame, it is highly recommended for multistory construction.

- The maximum and minimum amounts of reinforcement are limited to prevent rebar congestion and assure a nonbrittle flexural response. Although the precise limits are different for the two types of frames, a great portion of practical, buildable designs will satisfy either.

- Minimum amounts of continuous reinforcement to account for moment reversals are required by placing lower limits on the flexural strength at any cross section. Requirements for the two types of frames are similar.

- Locations for splices of reinforcement are more tightly controlled for the Special Frame.

- In addition, the Special Frame must satisfy numerous other requirements beyond the Intermediate Frame to assure that member proportions are within the scope of the present research experience on seismic resistance and that the analysis, the design procedures, the qualities of the materials, and the inspection procedures are at the highest level of the state of the art.

11.6 SEISMIC PERFORMANCE CATEGORY A

Construction qualifying under Category A as identified in Table 1-2 may be built with no special detailing requirements for earthquake resistance except for ties around anchor bolts as indicated in Sec. 11.1. "Closely enclosed" is intended to mean that the ties should be located within 3 to 4 bolt diameters of the bolts.

11.7 SEISMIC PERFORMANCE CATEGORY B

Special details for ductility and toughness are not required in Category B.

11.8 SEISMIC PERFORMANCE CATEGORY C

A frame used as part of the lateral force resisting system in Category C as identified in Table 3-2 (see Chapter 3) is required to have certain details that are intended to help sustain integrity of the frame when subjected to deformation reversals into the nonlinear range of response. Such frames must have attributes of Intermediate Moment Frames. Structural (shear) walls of buildings in Category C are to be built in accordance with the requirements of Ref. 11.1.

11.9 SEISMIC PERFORMANCE CATEGORIES D AND E

The requirements conform to current practice in the areas of highest seismic hazard.

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Chapter 12 Commentary

MASONRY

12.1 REFERENCE DOCUMENTS

This section references the recently completed *Building Code Requirements for Masonry Structures* (ACI-ASCE 530), which covers all types of masonry (clay, concrete, glass, stone, etc.). Construction and quality assurance requirements are included by reference to *Specifications for Masonry Structures* (ACI-ASCE 530.1). These design and construction documents reference nationally recognized testing standards and material standards developed by ASTM and others.

Concern has been expressed about the area of vertical reinforcement permitted in Sec. 3.1.2 of Ref. 12.1. The percentage of the area of the grout space (minimum grout area) and the cover and clearance requirements in Chapter 8 of Ref. 12.1 provide reasonable assurance that the strength of the reinforcement can be developed.

12.1.1 Modifications to Appendix A of Ref. 12.1

Appendix A provisions of ACI-ASCE 530, "Special Provisions for Seismic Design," are based on seismic zones defined by ANSI A58.1, *Minimum Design Loads for Buildings and Other Structures*. To be consistent with the *NEHRP Recommended Provisions*, Table 12-1 correlates seismic zones to seismic performance categories.

12.2 STRENGTH OF MEMBERS AND CONNECTIONS

The strength of members and connections is based on working stress procedures multiplied by a factor to approximate typical capacity. Capacity is approximated to equal the allowable stress determined by Ref. 12.1 multiplied by a 1.33 factor for load combinations that include wind or earthquake (Ref. 12.1, Sec. 5.3.2) and further multiplied by a 2.5 factor.

The resulting approximate capacity is 3.3 times the allowable stress. The design strength is equal to the approximated capacity times the capacity reduction factor, ϕ, to achieve a reliable design level value.

12.2.1 Splice length of reinforcement is based on the allowable stress in the reinforcement in accordance with Ref. 12.1, Sec. 8.5.7. This allowable stress is not modified by the 2.5 factor from Sec. 12.2 or by a ϕ factor. Splice lengths required by these provisions are therefore identical to the splice length required by Ref. 12.1.

12.3 RESPONSE MODIFICATION COEFFICIENTS

Masonry designed in accordance with Chapter 7 of Ref. 12.1 requires reinforcement to resist tension as well as minimum levels of reinforcement and detailing based on seismic zone (i.e., *NEHRP Recommended Provisions* Seismic Performance Category). These requirements are intended to provide a level of inelastic cyclic straining capacity consistent with the response modification coefficients of Table 3-2 for reinforced masonry. "Unreinforced" masonry shear walls designed in accordance with Chapter 6 of Ref. 12.1 that do not tolerate inelastic straining without loss of strength use lower response modification coefficients to ensure that "unreinforced" masonry shear walls remain within the elastic range when subjected to design level seismic forces. The term "unreinforced masonry" is not used in Ref. 12.1 but is considered analogous to masonry designed by Chapter 6 of Ref. 12.1, "Design Allowing Tensile Stresses in Masonry."

12.4 SEISMIC PERFORMANCE CATEGORY A

Ref. 12.1 permits three design methods for masonry:

- Design allowing tensile stresses in masonry (Chapter 6),
- Design neglecting tensile strength of masonry (Chapter 7), and
- Empirical design of masonry (Chapter 9).

Any of the three methods are considered appropriate for designs in Category A.

12.5 SEISMIC PERFORMANCE CATEGORY B

Masonry may be designed by Methods 1, 2, or 3 described above; however, in Category B, design of the lateral load resisting system must be based on analysis in accordance with Methods 1 or 2 described above.

12.6 SEISMIC PERFORMANCE CATEGORY C

In addition to the requirements of Category B, minimum levels of reinforcement and detailing are required in accordance with Appendix A of Ref. 12.1. Further, noncomposite wythes (i.e., cavity walls) and screen walls must meet the detailing requirements of Sec. 12.6.1.1 and Sec. 12.6.1.2, respectively.

12.7 SEISMIC PERFORMANCE CATEGORY D

In addition to the requirements of Category C, the area and spacing of shear reinforcement for shear walls must meet the requirements of Sec. 12.7.2. Special inspection is required in accordance with Sec. 1.6.2.5.

12.8 SEISMIC PERFORMANCE CATEGORY E

The additional requirements of Category E are intended to ensure that the structure remains functional after the earthquake.

Appendix A

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Blair Tulloch, Tulloch Construction, Inc., Oakland, California (representing the Associated General Contractors of America)

Ajit S. Virdee, CYGNA Consulting Engineers, Sacramento, California (representing the Structural Engineers Association of California)

Alan H. Yorkdale,[2] Brick Institute of America, Reston, Virginia

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[1]Member, Executive Committee.

[2]Deceased.

Appendix A

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[3]Member, Executive Committee.

Appendix A

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Appendix A

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TC 6 Representative

Robert D. Hanson, University of Michigan, Ann Arbor

TC 7 Representative

Edwin G. Zacher (TMC Chairman), H J Brunnier Associates, San Francisco, California

TC 8 Representative

John Fisher, Consultant, Portola, California

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[4]Meetings of the TMC: May 12-13, 1987, Denver, Colorado; June 29-30, 1987, Denver, Colorado; August 24-25, 1987, Denver, Colorado; October 15-16, 1987, Denver, Colorado; October 30-31, 1987, Denver, Colorado; November 24, 1987 (Conference Call); May 24-25, 1988, Denver, Colorado. Meeting of the TMC ad Hoc Committee on Mapping, September 2, 1987, San Francisco, California.

[5]Deceased.

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Harry W. Martin, American Iron and Steel Institute, Newcastle, California (January-August 1988)

Michael Sbaglia, American Insurance Services Group, New York, New York

Appendix A

TECHNICAL COMMITTEES

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TC 2, Structural Design[7]

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Nicholas F. Forell, Forell/Elsesser Engineers, Inc., San Francisco, California

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[6]Meetings: March 4, 1987, Denver, Colorado, and June 12, 1987, Denver, Colorado.

[7]Meetings: January 14-15, 1987, San Francisco, California; June 11-12, 1987, Denver, Colorado; July 16, 1987, San Francisco, California; October 6, 1987, San Francisco, California.

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Building Seismic Safety Council

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Earthquake Engineering Research Institute

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Structural Engineers Association of California

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[8]Meetings: January 14, 1987, San Francisco, California, and September 10, 1987, Berkeley, California.

[9]Meetings: January 14-15, 1987, San Francisco, California; June 12, 1987, Denver, Colorado; July 27, 1987, Denver, Colorado.

American Society of Civil Engineers

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Applied Technology Council

Sigmund Freeman, Wiss, Janney, Elstner Associates Inc., Emeryville, California

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Walter C. Oram (June 1987-August 1988), Concrete Reinforcing Steel Institute, Lawndale, California

Earthquake Engineering Research Institute

James O. Jirsa, University of Texas, Austin, Texas

Interagency Committee on Seismic Safety in Construction

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National Ready Mix Concrete Association

Jim Rossberg, National Ready Mix Concrete Association, Silver Spring, Maryland

Portland Cement Association

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Post Tensioning Institute

Neil M. Hawkins (Chairman), Univerity of Washington, Seattle

Prestressed Concrete Institute

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Alex Aswad (June 1987-August 1988), Pennsylvania State University, Harrisburg Middletown, Pennsylvania

Structural Engineers Association of California

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TC 5, Masonry[10]

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American Society of Civil Engineers

Daniel Shapiro, Shapiro, Okino, Hom and Associates, San Francisco, California

Brick Institute of America

Alan H. Yorkdale[11] (Representative to the TMC, January-October 1987), Brick Institute of America, Reston, Virginia

Interagency Committee on Seismic Safety in Constr.

Charles F. Scribner, National Bureau of Standards, Gaithersburg, Maryland

Masonry Institute of America

James E. Amrhein, Masonry Institute of America, Los Angeles, California

National Concrete Masonry Association

Mark B. Hogan (Representative to the TMC, October 1987-August 1988), National Concrete Masonry Association, Herndon, Virginia

Structural Engineers Association of California

Ben L. Schmid, B. L. Schmid, Consultant Structural Engineer, Pasadena, California

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[10]Meetings: January 5-6, 1987, Denver, Colorado; March 21-22, 1987, San Antonio, Texas; June 15-16, 1987, Reston, Virginia.

[11]Deceased.

The Masonry Society

John Tawresey (Chairman), KPFF Engineers, Seattle, Washington

Western States Clay Products Association

Donald A. Wakefield, Interstate Brick and Ceramic Tile, West Jordan, Utah

TC 6, Steel[12]

American Institute of Steel Construction

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American Iron and Steel Institute

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American Society of Civil Engineers

Horatio Allison, Allison, McCormac, and Nickolaus, Rockville, Maryland

Building Seismic Safety Council

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Interagency Committee on Seismic Safety in Construction

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Metal Buildings Manufacturers Association

Joe N. Nunnery, AMCA Buildings Division, Memphis, Tennessee

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[12]Meetings: January 14-15, 1987, San Francisco, California; June 11-12, 1987, Denver, Colorado; August 19, 1987, Orlando, Florida.

Structural Engineers Association of California

Egor P. Popov, University of California, Berkeley

TC 7, Wood[13]

National Forest Products Association

Thomas E. Brassell (Alternate Chairman and Alternate Representative to the TMC), American Institute of Timber Construction, Englewood, Colorado

American Plywood Association

Kenneth R. Andreason, American Plywood Association, Tacoma, Washington

American Society of Civil Engineers

Thomas G. Williamson, Lamfab Wood Structures, Indianapolis, Indiana

Building Seismic Safety Council

Edwin G. Zacher (Chairman and Representative to the TMC), H J Brunnier Associates, San Francisco, California

Interagency Committee on Seismic Safety in Construction

Billy Bohannan (January-June 1987), U.S. Forest Service Forest Products Laboratory, Madison, Wisconsin

Erwin L. Schaffer (June 1987-August 1988), U.S. Forest Service Forest Products Laboratory, Madison, Wisconsin

National Forest Products Association

Wallace A. Norum, National Forest Products Association, Mountain View, California

U.S. Forest Products Laboratory

Robert Falk, U.S. Forest Service Forest Products Laboratory, Madison, Wisconsin

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[13]Meetings: January 14-15, 1987, San Francisco, California, and-May 11, 1987, Denver, Colorado.

TC 8, Architectural, Mechanical, and Electrical Systems[14]

American Institute of Architects

John Fisher (Representative to the TMC), Consultant, Portola, California

American Society of Civil Engineers

Bruce C. Olsen (Chairman), Consulting Engineer, Seattle, Washington

American Society of Heating, Refrigeration and Air Conditioning Engineers

Patrick Lama, Mason Industries, Hauppauge, New York

Brick Institute of America

Gerald A. Dalrymple, Brick Institute of America, Reston, Virginia

Building Seismic Safety Council

Donn Harter, California Glass Association, Bellflower, California

Interagency Committee on Seismic Safety in Construction

Delano Surdahl, Albuquerque Operations Office, Department of Energy, Albuquerque, New Mexico

National Elevator Industry, Inc.

George W. Gibson, National Elevator Industry, Inc., Farmington, Connecticut

Sheet Metal and Air Conditioning Contractors' National Association

Walter Drown, Sheet Metal and Air Conditioning Contractors' National Association, Los Angeles, California

Structural Engineers Association of California

Eugene Cole, Cole, Yee, Schubert and Associates, Sacramento, California

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[14]Meetings: January 14-15, 1987, San Francisco, California, and June 11, 1987, Denver, Colorado.

Appendix A

TC 9, Regulatory Use[15]

Associated General Contractors of America

Cecil Mark, Mark Diversified, Sacramento, California

American Institute of Architects

David Bullen, American Institute of Architects, Washington, D.C.

American Society of Civil Engineers

Norton Remmer (Chairman), Consulting Engineer, Worcester, Massachusetts

Association of Major City Building Officials

Franklin Lew, City of San Francisco, California

Building Officials and Code Administrators International

Bob McCluer, Bldg Officials and Code Administrators International, Country Club Hills, Illinois

Building Seismic Safety Council

Vincent R. Bush (Representative to the TMC), Consultant, Walnut, California

Interagency Committee on Seismic Safety in Construction

G. Robert Fuller, Department of Housing and Urban Development, Washington, D.C.

International Conference of Building Officials

Phillip C. Phillips, City of San Leandro, California

National Council of States on Building Codes and Standards

David Conover, National Council of States on Building Codes and Standards, Herndon, Virginia

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[15]Meetings: January 14-15, 1987, San Francisco, California; June 11-12, 1987, Denver, Colorado; July 22, 1987, Denver, Colorado.

Southern Building Code Congress International

John R. Battles, Southern Building Code Congress International, Birmingham, Alabama

Appendix B

SUMMARY OF DIFFERENCES BETWEEN THE 1985 AND 1988 EDITIONS OF THE *NEHRP RECOMMENDED PROVISIONS*

The substantive differences between the 1985 and 1988 Editions of the *NEHRP Recommended Provisions*, which are highlighted in the margins of the document, are summarized below.

CHAPTER 1, GENERAL PROVISIONS

Intent of the *Provisions*

Both the *Provisions* and the *Commentary* volumes have been modified to clarify and emphasize the intent of the *Provisions*.

In the 1988 Edition of the *Provisions* volume, Sec. 1.1, "Purpose," makes specific reference to the different levels of resistance required for various types of structures, a matter that had not been covered in the 1985 Edition. The text also distinguishes between expected building performance during the "design earthquake" (which may result in some structural and nonstructural but repairable damage) and during a "maximum credible" earthquake (which may result in considerable damage but a low likelihood of building collapse), a distinction that was not clear in the 1985 Edition.

Commentary Sec. 1.1 also has been revised extensively to address in considerable detail many of the philosophical issues behind development of the *Provisions*. Thus, the 1988 version of this section explains that:

- The *Provisions* provide the minimum criteria considered "prudent and economically justified."
- The *Provisions* are applicable throughout the United States.
- The *Provisions* have been developed under a consensus balloting procedure and therefore may be reasonably adopted into building codes.

- The *Provisions*, unlike some other seismic design documents, attempt to limit property damage only to the extent that it affects life safety and emergency recovery efforts.

- Structural and nonstructural damage to a building designed using the *Provisions* is to be expected during a "design earthquake," but the damage is expected to be repairable.

- Damage during a more severe or intense earthquake may be much more extensive than during a "design earthquake" but the likelihood of collapse will be small.

- Judgment on the part of the design engineer is needed to account for the many variables that affect a project (e.g., the dynamic characteristics of the structure and the variability in ground motion, earthquake intensity, distance from the earthquake epicenter, and soil type).

- The "design earthquake" identified by the *Provisions* has a 90 percent probability of not being exceeded in 50 years (or, alternatively, a 10 percent probability of being exceeded in 50 years). In the central and eastern United States, however, the "maximum intensity" earthquake may be considerably greater than the "design earthquake" and maps based on a 90 percent probability of not being exceeded in 250 years (or, alternatively, a 10 percent probability of being exceeded in 250 years) have been added to the 1988 Edition to give users a basis for addressing this problem.

- If damage control is desired, the designer must recognize that deformations larger than the drift limits specified in the *Provisions* may occur during an earthquake larger than the "design earthquake." Thus, the design should provide not only sufficient strength to resist the specified seismic loads but also the proper stiffness to limit lateral deformation.

- In cases where wind loads control stress or drift design, special requirements for the seismic resisting system still must be followed to ensure sufficient energy absorption capacity in the structure.

- Providing a continuous load path is vital to ensuring desired building response for structures designed using lateral force design provisions.

In the 1988 Edition of the *Commentary*, Sec. 1.2, "Scope," explains that the *Provisions* document does not address existing buildings. The 1985 Edition had included the Applied Technology Council's 1978 recommendations (from ATC 3-06) for existing buildings as Part 3 of the document simply for the information of those interested and it was noted that those recommendations had not received formal consideration and approval by the Council. In fact, these provisions had been effectively segregated from provisions for new buildings many years earlier and this, coupled with the fact that FEMA is sponsoring work that will lead to up-to-date provisions for existing buildings, stimulated deletion of the existing building information from the 1988 Edition of the *Provisions*.

The 1988 version of *Commentary* Sec. 1.2 also emphasizes that the *Provisions* document is intended for use only in building design and that it does not apply to non-building structures such as dams, bridges, and power plants. It also specifically states that the *Provisions* adresses only earthquake ground-shaking and not problems of liquefaction, settlement, slides, subsidence, and faulting.

Risk Maps

The four seismic risk maps (Figures 1-1 through 1-4) included in the 1985 Edition of the *Provisions* are used again in the 1988 Edition; they define the seismic ground-shaking hazard in terms of an acceleration coefficient (A_a) and a velocity-related acceleration coefficient (A_v) for exposure times of 50 years (with a 10 percent probability of exceedance). The 1988 Edition, however, also presents a new procedure and four new maps (Figures 1-5 through 1-8) that define the seismic ground-shaking hazard in terms of acceleration (A) and velocity (v); Figures 1-5 and 1-6 are for exposure times of 50 years and Figures 1-7 and 1-8 are for exposure times of 250 years (all four with a 10 percent probability of exceedance). Since the *Provisions* makes use of velocity-related acceleration for calculating design forces and determining the applicability of certain requirements, a procedure for directly using the mapped velocities in centimeters per second was developed for use of the new maps. The new procedure and maps, which are presented in the "Appendix to Chapter 1," are offered as an alternative to the original procedure and maps and their evaluation through use is encouraged. The "Appendix to Chapter 1" describes the new procedure by highlighting the changes that need to be made in specific parts of the *Provisions* so that the new maps can be used.

The new maps incorporate advances in seismic risk mapping including consideration of uncertainty in ground motion attenuation and fault rupture length. They also address the issue of truncation of, or placing an artificial cap on, high acceleration and velocity values. Figures 1-1 through 1-4 reflect a truncation of high values of ground acceleration but, since it was performed in the mapping process itself, it is not apparent to the user of the *Provisions*. Figures 1-5 through 1-8 also have been truncated but at a significantly higher value and, thus, they better reflect the calculated differences in accelerations and velocities for an equal level of probability. Nevertheless, the use in design of values over certain levels is not recommended and the

"Appendix to Chapter 1" provides users of the new maps with procedures for reducing any mapped values over these levels. This reduction is based in part on experience with modern buildings in recent earthquakes, in part on the concept that the highest spikes of short duration acceleration are not transmitted uniformly across the soil-foundation interface to large objects such as buildings, and on other concepts. It should be noted, however, that the truncation concept remains somewhat controversial and in need of further research and study.

Changes to Existing Buildings

As mentioned above, Sec. 1.2 of the *Commentary* was rewritten to indicate that existing buildings are not covered by the *Provisions*, and a paragraph is added to Sec. 1.3, "Application of the *Provisions*," to indicate that alterations and repairs also are not covered.

The *Provisions* do, however, cover additions to existing buildings and changes in use, and Sec. 1.3 was revised for the 1988 Edition to clarify and define the conditions under which the *Provisions* apply in these cases.

It is stated that new additions are required to be designed and constructed in accordance with the *Provisions*, but that the relationship of the new addition to the existing building determines whether the existing building must be brought into conformance with the *Provisions*. When the addition is structurally independent and does not reduce the seismic resistance of the existing building, no changes are required in the existing building. When a structurally dependent addition is planned, the existing structure must meet the requirements of the *Provisions* if the addition will cause seismic forces in the existing building to be increased by more than a nominal amount or if the new work reduces the seismic resistance of the existing building.

It also is stated that when a change in use of an existing building results in a higher Seismic Hazard Exposure Group, the existing structure must be brought into accordance with the *Provisions* requirements for new buildings.

Seismicity Index, Seismic Performance Category and Seismic Hazard Exposure Group

The 1988 Edition of the *Provisions* no longer uses the Seismicity Index, a change that actually is entirely editorial and does not affect the development of the design forces or any other design matter. In the 1985 Edition, the user was required to select the appropriate acceleration, A_a, or velocity-related acceleration, A_v, from the maps (Figures 1-1 through 1-4) and then determine the Seismicity Index from the value for A_v. The Seismicity Index then was used in a table relating it to Seismic Hazard Exposure Group in order to determine the Seismic Performance Category. Elimination of the Seismicity Index in the 1988 Edition simply eliminates one step in this cumbersome process and permits the user to determine the Seismic Performance Category from a table (Table 1-2) that relates it directly to Seismic Hazard Exposure Group and A_v (or velocity, v, in the "Appendix to Chapter 1").

The number of Seismic Performance Categories was expanded from four to five in the 1988 Edition, another change that is more editorial than substantive. The 1985 Edition had been issued with the chapters on concrete and masonry dividing Seismic Performance Category B into two subcategories, B.1 and B.2. The 1988 Edition's use of five categories (A, B, C, D, and E) is meant to simplify and unify the categories in use. The old and new Seismic Performance Categories are related as shown below:

1985 Edition Category	1988 Edition Category
A	A
B.1	B
B.2	C
C	D
D	E

Chapters 7, 9, and 10 on foundations, wood and steel, respectively, did not distinguish between Categories B.1 and B.2 in the 1985 Edition, and they generally have simply been edited to refer to Seismic Performance Categories B and C where before they referred to Category B. The remainder of the 1988 Edition reflects editorial changes made to incorporate the new Seismic Performance Category designations.

The definitions of the Seismic Hazard Exposure Groups (Sec. 1.4.2) have been refined for the 1988 Edition of the *Provisions*. Additional examples of Seismic Hazard Exposure Group III facilities, buildings having essential facilities necessary for post-earthquake recovery, are included, and examples of Seismic Hazard Exposure Group II facilities, buildings that constitute a substantial public hazard because of their occupancy or use, are tied more closely to numbers of occupants to more accurately reflect the intent of the *Provisions* as described in the *Commentary* and to more closely align the Seismic Hazard Exposure Groups with the model code occupancy categories.

Access requirements for Seismic Hazard Exposure Group III buildings also are more specifically defined (Sec. 1.4.2.5), and the functionality requirement (Sec. 1.4.2.6) is relaxed (i.e., it has been changed from "shall" to "shall, in so far as practical").

Quality Assurance and Inspection Procedures

The paragraphs defining which buildings require a formal Quality Assurance Plan (Sec. 1.6.1) have been rewritten to clarify and slightly relax the requirements in order to make the *Provisions* more consistent with the needs and practices of building officials throughout the United States. The new definition is based on Seismic Performance Category rather than on the combination of Seismic Hazard Exposure Group and Seismicity Index used in the 1985 Edition. A Quality Assurance Plan for the Designated Seismic Systems is required for buildings in Seismic Performance Category E whereas such a plan is required only for the Structural Seismic Resisting Systems of Buildings in Seismic Performance Categories C and D.

A change in terminology from "Special Inspector" to "Owner's Inspector" is introduced in Sec. 1.6.2 of the 1988 Edition to emphasize the role and responsibility of this individual.

Requirements for continuous special inspection were relaxed for structural masonry (Sec. 1.6.2.5) and single pass fillet welds in structural steel (Sec. 1.6.2.6).

In the 1988 Edition, the requirements for special testing of masonry now reference ACI-ASCE 530.

Regulatory Agency

In order to simplify the 1988 Edition, the term "Regulatory Agency" is used throughout instead of the variety of terms (e.g., "authority having jurisdiction" and "Cognizant Jurisdiction") that was used in the 1985 Edition.

CHAPTER 2, DEFINITIONS AND SYMBOLS

In addition to the editorial revisions needed to reflect other changes made in the 1988 Edition of the *Provisions*, definitions were added to Chapter 2 for the following terms: base shear, story drift ratio, and story shear. Definitions for braced frame, concentrically braced frame, eccentrically braced frame (with subcategories for link beams, link beam rotation angle, link beam shear yield strength, diagonal brace, lateral support members, link beam end web stiffeners, and link beam intermediate web stiffeners) were also added to reflect the integration of the 1985 Edition's "Appendix to Chapter 10" into the body of the *Provisions* (see the discussion of Chapter 10 changes below).

CHAPTER 3, STRUCTURAL DESIGN REQUIREMENTS

Soil Profile Types

A fourth soil profile type, S_4, has been added to Sec. 3.2 in the 1988 Edition to account for the type of soil response observed during the 1985 Mexico City earthquake. Soil Profile Type S_4 is described as a profile with more than 70 feet of soft clays or silts. Section 1.4 of the *Commentary* describes the Mexico City experience that supports the inclusion of this fourth soil profile type.

R and C_d factors

Table 3-2, "Response Coefficients," has been reformatted for the 1988 Edition to make it easier to understand and use. Clarification of the definition and applications of dual systems was of special concern in the design of the modified table. R and C_d factors for eccentrically braced frames were added to the table to reflect the Chapter 10 change that moved the design provisions for these frames from an appendix into the body of the chapter. The R and C_d factors for eccentrically braced frames were adjusted to reflect modifications that have been found to be appropriate during the several years they have been in use.

Building Configuration

The 1988 Edition defines plan and vertical irregularities in detail in the *Provisions* and discusses them in the *Commentary* whereas the 1985 Edition addressed the topic in depth only in the *Commentary*. This has been accomplished primarily by adding Table 3-3, "Plan Structural Irregularities," and Table 3-4, "Vertical Structural Irregularities." These tables list specific building features that create irregularity in a building. The existence of any one of these features makes the building irregular, with certain exceptions and limitations. The tables, exceptions, and limitations are adapted from similar tables and exceptions developed by the Structural Engineers Association of California for the 1988 Edition of the *Uniform Building Code*.

Tables 3-3 and 3-4 describe five categories of plan irregularity and five categories of vertical irregularity and refer the user to the sections of the *Provisions* related to the design of a building with the given irregularity. In some cases, more accurate dynamic analysis of the building is required whereas in other cases, additional prescriptive detailing or proportioning is necessary. Some irregularities are acceptable only in small buildings. Most of the restrictions and requirements for irregular buildings apply only to the higher Seismic Performance Categories.

Drift Limits

Table 3-5, "Allowable Story Drift," has been revised for the 1988 Edition to clarify and consolidate Sec. 3.8 and Table 3-C of the 1985 Edition. The basic limits did not change, but the exceptions and limitations were somewhat modified.

CHAPTER 4, EQUIVALENT LATERAL FORCE PROCEDURE

Weight

The definition of W, weight of the structure to be included in seismic base shear calculations, in Sec. 4.2, "Seismic Base Shear," has been rewritten to clarify the intent of the *Provisions*. An additional requirement was added to bring the *Provisions* in conformance with standard practice such as is described in the *Uniform Building Code* (i.e., minimum partition loads of 10 psf must be included in W when a partition load allowance is included in floor design).

Period

An alternate equation was included for determining T, the fundamental building period, in Sec. 4.2.2, "Period Determination." The equation, $T_a = 0.1N$, with limitations on story height and on N (number of stories), is an older equation that has been in use for many years but that was not included in the 1985 Edition of the *Provisions*. Recent research reaffirmed the applicability of this equation and provided the motivation for including it in the 1988 Edition.

Torsion

Improved procedures for handling torsion that appear in the 1988 *Uniform Building Code* have been adapted for inclusion in Sec. 4.4.1, "Torsion," of the 1988 Edition of the *Provisions* as well. The new procedure requires that the accidental torsion be amplified at each level by an amplification factor A_x for Seismic Performance Categories C, D, and E when a torsional irregularity exists. A formula for determining A_x for each story (Eq. 4-8) and a maximum limit on A_x are part of the new procedure.

P-Delta

A warning has been added to the *Commentary* for Sec. 4.6, "Drift Determination and P-delta Effects" to acknowledge that the procedure in Sec. 4.6 of the *Provisions* may not be conservative in some instances. A ballot proposal to change the stability equations to use the secant stiffness (by deleting the C_d term from the denominator of Eq. 4-11) was overturned during the reballot. The warning added to the *Commentary* points out that both the published procedure and the secant stiffness procedure are based on static stability and that dynamic stability is a far more complex problem.

CHAPTER 5, MODAL ANALYSIS PROCEDURE

No changes were made to Chapter 5 for the 1988 Edition of the *Provisions*, but minor revisions were made to the *Commentary* to reflect the clarified intent of the *Provisions*.

CHAPTER 6, SOIL-STRUCTURE INTERACTION

Chapter 6 of the 1985 Edition has been relegated in its entirety to an appendix (the "Appendix to Chapter 6") in the 1988 Edition. Use of the detailed procedures on soil-structure interaction was judged to be unnecessary for most structures, and it was concluded that the 1985 Edition's Chapter 6 requirements were too specialized for inclusion in the body of the 1988 Edition of the *Provisions*.

CHAPTER 7, FOUNDATION DESIGN REQUIREMENTS

Editorial revisions (some suggested late in the 1985 balloting procedure) were incorporated in the 1988 Edition of the *Provisions* to clarify the intent of Chapter 7. One of the major editorial changes involved clarifying the distinction between concrete piles, concrete-filled pipe piles, and metal-cased concrete piles.

The requirements for ties between individual foundation elements were relaxed somewhat in the 1988 Edition (i.e., reinforced concrete slabs on grade may be used as ties and hard cohesive soils and very dense granular soils may be considered to confine the foundation elements).

Reinforcement requirements for metal cased concrete piles were modified to allow the metal casing to be used as confinement if it meets certain minimum thickness and corrosion protection requirements. New requirements for minimum ties or spirals were added for precast concrete piles.

For Seismic Performance Categories D and E, new requirements were added stipulating that all piles must be able to withstand the curvatures induced by the seismic forces, a requirement that only applied to precast concrete piles in the 1985 Edition. As a result of the new requirements, procedures for determining minimum spiral reinforcement in precast-prestressed piles subject to severe forces are omitted from the 1988 Edition and the use of precast-prestressed piles in the highest Seismic Performance Categories (which was prohibited in the 1985 Edition) is allowed.

CHAPTER 8, ARCHITECTURAL, MECHANICAL AND ELECTRICAL COMPONENTS AND SYSTEMS

In Chapter 8, the word "attachment" is used exclusively in the 1988 Edition rather than being used interchangeably with the word "connector" as in the 1985 Edition. This change was made to help distinguish nonstructural connections (attachments) from structural connections.

A more complete derivation of the equations in Sec. 8.2, "Architectural Design Requirements," has been added in the 1988 Edition of the *Commentary*.

Other changes include a new section dealing with raised access floors (Sec. 8.2.6) and a requirement for flexible connections at utility-structure interfaces (Sec. 8.3.5.2).

CHAPTER 9, WOOD

Section 9.1, "Reference Documents," has been updated for the 1988 Edition to include the most recent versions of the standards and specifications that apply to design in wood.

The values for plywood diaphragms in Table 9-1, "Allowable Shear in Pounds per Foot (at Working Stress) for Horizontal Diaphragms with Framing Members of Douglas Fir-Larch or Southern Pine for Seismic Loading," also have been updated to include the most recent nationally accepted values.

CHAPTER 10, STEEL

Reference Documents

The listing of reference documents for steel (Sec. 10.1) has been updated for the 1988 Edition. Where changes in the new editions of the reference documents affected the *1985 Edition of the Provisions*, the corresponding changes have been made for the 1988 Edition.

Added to the list is the American Institute of Steel Construction's (AISC's) *Load and Resistance Factor Design Specification for Structural Steel Buildings* (September 1, 1986). To reflect the new reference, load and resistance factor design (LRFD) is presented in the 1988 Edition of the *Provisions* in a new "Appendix to Chapter 10" as an alternate design procedure. The only modification to the LRFD procedure as specified in the AISC document is that for Seismic Performance Categories D and E, detailing that will provide results equivalent to those obtained by the use of the provisions in the body of Chapter 10 is required.

Braced Frames

A new section (Sec. 10.8) dealing with concentrically braced frames has been added for the 1988 Edition to apply to all bracing systems other than eccentrically braced frames. The changes in design rules for concentric bracing systems are substantial; however, they apply only to Seismic Performance Categories D and E. These new requirements are included in the 1988 Edition to reflect the current state of technical knowledge.

In the 1985 Edition of the *Provisions*, the design of eccentrically braced frames was covered in an appendix to Chapter 10. Since these provisions have been reviewed and validated at many levels during the past three years, they have been incorporated into the body of Chapter 10 in the 1988 Edition. Refinements to the eccentrically braced frame requirements that have been made since publication of the 1985 Edition were included where appropriate.

As in other chapters, editorial changes were made, some in response to 1985 ballot comments, to clarify the intent of the *Provisions* and to accommodate the other changes made in the document.

CHAPTER 11, REINFORCED CONCRETE

Changes to Chapter 11 include refinement of the modifications to the basic reference document, the American Concrete Institute's *Building Code Requirements for Reinforced Concrete* (ACI 318-83). These changes included the following:

- The prohibition on welding of stirrups, etc., to longitudinal reinforcing was dropped. The basic reference standard (ACI-318) requires the specific approval of the engineer for such welding.

- The additional requirements for transverse reinforcement were refined to specifically address placement at walls and at footings.

- The 1985 modification of force calculation requirements in structural truss members, struts, ties, and collector members (Sec. A.5.2.3 of ACI 318-83) to "may" was revoked; the 1988 Edition wording reverts to "shall."

- Hooking of transverse reinforcement terminating at edges of shear walls is not required in the 1988 Edition when stresses drop below a certain level.

- The definition of joint area A_j was reworded.

- The modification requiring that shear strength of the concrete section be ignored when the earthquake-induced shear represents more than 1/4 of the total shear was dropped. Thus, the requirement reverts to the less stringent requirement of ACI 318 based on earthquake shear of more than 1/2 the total shear.

- Precast and other systems not specifically meeting the requirements of Appendix A of ACI 318-83 may be used if it can be shown by evidence and analysis that comparable strength and toughness will be achieved.

- Post-tensioning tendons may be used in seismic resisting elements within certain limitations and additional requirements on the use of post-tensioning tendons are specified.

- Equation A-4 in ACI 318 for the minimum area of rectangular hoop reinforcement required is modified; 0.12 is reduced to 0.09.

- Maximum allowable spacing of transverse reinforcement in joints is increased to 6 inches under certain circumstances.

As for other chapters, editorial changes were made as necessary to reflect the changes made in other areas of the document (e.g., Table 11-B which defined Class B.1 and B.2 in the 1985 Edition was omitted since the change to five Seismic Performance Categories in the 1988 Edition eliminated the need for it).

Changes to the *Commentary* of Chapter 11 include the addition of a discussion of and a list of references that address the design of seismically resistant precast structures.

CHAPTER 12, MASONRY

Chapter 12 has been completely rewritten for the 1988 Edition of the *Provisions*. The 1985 Edition referenced several standards and required use of the *Uniform Building Code* masonry provisions for Seismic Performance Categories C and D. The 1988 Edition references instead the new American Concrete Institute-American Society of Civil Engineers *Building Code Requirements for Masonry Structures* (ACI-ASCE 530) including Appendix A, "Special Provisions for Seismic Design," and *Specification for Masonry Structures* (ACI-ASCE 530.1-88). This reference is the first national consensus standard for masonry developed in nearly 30 years. The *Commentary* for Chapter 12 also was rewritten to incorporate the change in design procedures.

The bulk of the technical requirements in the 1988 Edition are the same as or are very similar to those given in the 1985 Edition, but their appearance is different due to the fact that many of the requirements now are found in ACI-ASCE 530, either in the main body or in its seismic appendix. For example, the provisions of the 1985 Edition's "Appendix to Chapter 12" regarding anchor bolts in masonry are contained in the main body of ACI-ASCE 530. Most of the restrictions on materials and minimum reinforcement requirements are in the seismic appendix of ACI-ASCE 530.

Appendix C

THE BSSC PROGRAM
ON IMPROVED SEISMIC SAFETY PROVISIONS

BACKGROUND

Regulation of the design and construction of buildings in the United States has historically had as its principal aim the protection of public health and safety and, specifically, protection of the public from the actions of the individual property owner. In recent years, however, regulatory attention has been given to a growing array of public welfare issues such as the economic and social community impacts of large-scale property losses due to natural or manmade disasters.

In the case of earthquake hazard mitigation, the federal government is responsible for the performance of federal buildings and for limiting the financial loss exposure that stems from the President's authority to declare disaster areas and to provide a wide range of post-disaster services and assistance. Except for certain types of facilities, however, the federal government does not have the authority to prescribe standards affecting nonfederal buildings.

The Building Seismic Safety Council (BSSC) was conceived as an entirely new type of instrument for dealing with this complex regulatory environment and the related technical, social, and economic issues involved in developing and promulgating building earthquake hazard mitigation regulatory provisions that are national in scope. By bringing together in the BSSC all of the needed expertise and all relevant public and private interests, it was believed that the issues related to the seismic safety of buildings could be resolved and the jurisdictional problems overcome through authoritative guidance and assistance backed by a broad consensus.

The BSSC was established in 1979 under the auspices of the National Institute of Building Sciences (NIBS). It is an independent, voluntary membership body representing a wide variety of building community interests. Its fundamental purpose is to enhance public safety by providing a national forum that fosters improved seismic safety provisions for use by the building community in the planning, design, construction, regulation, and utilization of buildings.

To fulfill its purpose, the BSSC:

- Promotes the development of seismic safety provisions suitable for use throughout the United States;
- Recommends, encourages, and promotes the adoption of appropriate seismic safety provisions in voluntary standards and model codes;
- Assesses progress in the implementation of such provisions by federal, state, and local regulatory and construction agencies;
- Identifies opportunities for improving seismic safety regulations and practices and encourages public and private organizations to effect such improvements;
- Promotes the development of training and educational courses and materials for use by design professionals, builders, building regulatory officials, elected officials, industry representatives, other members of the building community, and the public;
- Advises government bodies on their programs of research, development, and implementation; and
- Periodically reviews and evaluates research findings, practices, and experience and makes recommendations for incorporation into seismic design practices.

The BSSC's area of interest encompasses all building types, structures, and related facilities and includes explicit consideration and assessment of the social, technical, administrative, political, legal, and economic implications of its deliberations and recommendations. The BSSC believes that the achievement of its purpose is a concern shared by all in the public and private sectors; therefore, its activities are structured to provide all interested entities (i.e., government bodies at all levels, voluntary organizations, business, industry, the design profession, the construction industry, the research community, and the general public) with the opportunity to participate. The BSSC also believes that the regional and local differences in the nature and magnitude of potentially hazardous earthquake events require a flexible approach to seismic safety that allows for consideration of the relative risk, resources, and capabilities of each community.

The BSSC is committed to continued technical improvement of seismic design provisions, assessment of advances in engineering knowledge and design experience, and evaluation of earthquake impacts. It recognizes that appropriate earthquake hazard reduction measures and initiatives should be adopted by existing organizations and institutions and incorporated, whenever possible, into their legislation, regulations, practices, rules, codes, relief procedures, and loan requirements so that these measures and initiatives become an integral part of established activities, not additional burdens. The BSSC itself assumes no standards-making and -promulgating role; rather, it advocates that code- and standards-formulation organizations consider BSSC recommendations for inclusion into their documents and standards.

PROGRAM FOR IMPROVED SEISMIC SAFETY PROVISIONS

It is in this context and with funding from the Federal Emergency Management Agency (FEMA) that the BSSC initiated its multiphased Program on Improved Seismic Safety Provisions directed toward the creation of authoritative, technically sound resource documents that can be used by the voluntary standards and model code organizations, the building community, the research community, and the public as the foundation for improved seismic safety design provisions.

The genesis of the effort of which the BSSC program is a major element began with initiatives taken by the National Science Foundation (NSF) as a part of its earthquake research support program. Under agreement with the National Bureau of Standards (NBS), the *Tentative Provisions for the Development of Seismic Regulations for Buildings* (referred to in this report as the *Tentative Provisions*) was prepared by the Applied Technology Council (ATC). As the ATC noted, the document was the product of a "cooperative effort with the design professions, building code interests, and the research community." Its purpose was to "...present, in one comprehensive document, the current state of knowledge in the fields of engineering seismology and engineering practice as it pertains to seismic design and construction of buildings." The document included many innovations, however, and ATC acknowledged that a careful assessment was needed.

Following the issuance of the *Tentative Provisions* in 1978, NBS released Technical Note 1100, "Analysis of Tentative Seismic Design Provisions for Buildings." In this note, NBS reported its "...systematic analysis of the logic and internal consistency of [the *Tentative Provisions*]." Based on its determination of the need to deal with unresolved public comments on the *Tentative Provisions* and issues raised in its own analysis, NBS issued a *Plan for the Assessment and Implementation of Seismic Design Provisions for Buildings* in November 1978 as its final submission to NSF. This plan included the following tasks:

1. A thorough review of the *Tentative Provisions* by all interested organizations;

2. The conduct of trial designs to establish the technical validity of the new provisions and to predict their economic impact;

3. The establishment of a mechanism to encourage consideration and adoption of the new provisions by organizations promulgating the appropriate national standards and model codes; and

4. Educational, technical, and administrative assistance to facilitate implementation and enforcement.

During this same period, other events significant for this effort were taking place. In October 1977, Congress passed the Earthquake Hazards Reduction Act of 1977 (P.L. 95-124) and the National Earthquake Hazards Reduction Program (NEHRP) was released by the Administration on June 22, 1978. The concept of an independent agency to coordinate all emergency management functions at the federal level also was under discussion. When this concept was effected and FEMA was created, FEMA became the implementing agency with NSF retaining its research-support role. Thus, the future disposition of the *Tentative Provisions* and the 1978 NBS plan shifted from NSF to FEMA.

The emergence of FEMA as the agency responsible for implementation of P.L. 95-124 (as amended) and the NEHRP also required the establishment of a mechanism for obtaining a broad public and private consensus on both recommended improved housing and building design and construction regulatory provisions and the means to be used in their promulgation. A series of meetings was held between representatives of the original participants in the NSF-sponsored project on seismic design provisions (the ATC; its parent organization, the Structural Engineers Association of California; NSF; and NBS), FEMA, the American Society of Civil Engineers, and the National Institute of Building Sciences. From these meetings, the concept of the Building Seismic Safety Council was born. As the concept began to take form, progressively wider public and private participation was sought, culminating in a broadly representative organizing meeting in the spring of 1979 in St. Louis, Missouri, at which time a charter and organizational rules and procedures were thoroughly debated and agreed upon.

In essence, the BSSC provided the mechanism envisioned in Task 3 of the 1978 NBS plan (a forum for elements of federal, state, and local governments; standards and model code bodies; professional societies; building industry and trade organizations; the research community; and consumer groups to participate equally in the development of a consensus on improved building seismic safety provisions and the means for their promulgation through the existing public and private building regulatory systems throughout the nation). A joint BSSC-NBS committee then was formed to conduct the review called for in Task 1 of the plan. The review effort concluded in 1980 and resulted in 198 recommendations for changes in the *Tentative Provisions (Review and Refinement of ATC 3-06 Tentative Seismic Provisions*, NBSIR 802111-11). FEMA provided funds to both the BSSC and NBS to support this activity.

As the review effort drew to a close, the BSSC and NBS created another joint committee (Committee 10A) to develop criteria by which the trial designs called for in Task 2 of the 1978 NBS plan could be evaluated and to recommend a specific trial design program plan. Subsequently, the BSSC created a special BSSC-NBS Trial Design Overview Committee (Committee 12) and charged it to, among other activities, revise the Committee 10A plan to accommodate a multiphased effort and to refine the *Tentative Provisions*, to the extent practicable, to reflect the recommendations generated during the earlier review. The Overview Committee completed the revised plan in August 1982. It was released in November 1982 as *Plan for a Trial Design Program To Assess Amended ATC 3-06 Tentative Provisions for the Development of Seismic Regulations for Buildings* (NBSIR 82-2589/BSSC 82-1).

THE TRIAL DESIGN EFFORT

The BSSC then initiated the effort to develop the actual trial designs, which would include the following building types and structural systems:

<u>Building Types</u>

Low-, mid-, and high-rise residential (R) buildings,
Mid- and high-rise office (O) buildings,
One-story industrial (I) buildings, and
Two-story commercial (C) buildings.

<u>Structural Systems</u>

1. Lateral load systems

   a. Shear walls
      (1) Cast-in-place concrete
      (2) Precast and prestressed-precast concrete
      (3) Masonry
      (4) Plywood on wood studs

   b. Braced frames--conventional steel

   c. Unbraced frames
      (1) Cast-in-place concrete both special and ordinary (as defined in the amended *Tentative Provisions*)
      (2) Steel, both special and ordinary, conventional and pre-engineered

2. Vertical load systems

   a. Bearing wall buildings
      (1) Walls
         (a) Cast-in-place concrete
         (b) Precast and prestressed-precast concrete
         (c) Masonry
         (d) Plywood on wood studs
      (2) Floors
         (a) Concrete slabs, both cast-in-place and precast, ordinary and prestressed
         (b) Steel joists with decks and slabs
         (c) Wood framing with plywood decks and lightweight concrete fill

   b. Framed buildings
      (1) Cast-in-place concrete flat slabs, waffle slabs, pan joists, and beam and slab systems, both ordinary and prestressed
      (2) Precast concrete, both ordinary and prestressed
      (3) Steel girder and purlin, beam and joist, and long-span truss systems with decks and slabs
      (4) Wood framing

It originally was intended that the trial design effort would be conducted in two phases with Phase I including trial designs for 27 new buildings in 4 major cities with medium to high seismic risk (Los Angeles, Seattle, Memphis, and Phoenix) and Phase II including 73 buildings in 7 cities (Los Angeles, Seattle, Memphis, Phoenix, New York, Chicago, and Minneapolis). Financial limitations, however, required that the program be scaled down as follows: During Phase I of the program, 10 design firms were retained to prepare trial designs for 26 new buildings in 4 cities with medium to high seismic risk--10 in Los Angeles, 4 in Seattle, 6 in Memphis, and 6 in Phoenix. During Phase II, 7 firms were retained to prepare trial designs for 20 buildings in 5 cities with medium to low seismic risk--3 in Charleston (S.C.), 4 in Chicago, 3 in Ft. Worth, 7 in New York, and 3 in St. Louis. For six of these buildings, alternative designs also were developed. (The schedule of basic designs from which the trial designs were chosen and the trial design firms are presented as Tables C-1 and C-2, respectively.)

Each building was designed twice: once according to the amended *Tentative Provisions* and once according to the prevailing local code for the particular location of the design. Basic structural designs (complete enough to assess the cost of the structural portion of the building), partial structural designs (special studies to test specific parameters, provisions, or objectives), and partial nonstructural designs (complete enough to assess the cost of the nonstructural portion of the building) were prepared and design and construction cost estimates were developed. The BSSC-NBS Overview Committee, assisted by a technical

TABLE C-1 Schedule of Basic Trial Designs

Plan Form	Vertical Load System	Seismic Resisting System Components	Other Vertical Components	Floor or Roof Components	Bldg. No.	No. of Stories	Los Angeles	Seattle	Memphis	Phoenix	New York	Chicago	Ft. Worth	Charleston	St. Louis
							Cities: Phase I				Phase II				
Residential	Bearing Walls	Plywood wall		Wood + plywood diaphragm	1	3	•	•							
		Concrete masonry wall		Wood + plywood diaphragm	2	3				•					
		Brick and concrete masonry		Wood + plywood diaphragm	2A	3						•			
		Concrete masonry wall		Prestressed slab	3	5				•			•		
		Brick wall		Reinforced concrete slab	4	5									
				Reinforced concrete slab	5	12	•				•				
				Reinforced concrete slab	5A	10									•
		Brick and concrete masonry wall		Steel joist	6	5								•	
				Steel joist	7	12									
		Reinforced conrete wall		Reinforced concrete slab	8	5			•						
				Reinforced concrete slab	9	12						•			
				Post-tensionsed slab	10	5				•					
		Precast concrete wall		Prestressed slab	11	5									
				Prestressed slab	12	12									
	Complete Vertical Load Carrying Frame	Steel, braced frame (transverse)/ moment frame (longitudinal)	Steel Framing	Steel joists	13	10									
			Steel Framing	Steel beam & RC slab	14	20			•						
		Reinforced concrete shear wall	RC framing	RC flat plate	15	10	•						•		
			RC framing	Post-tensioned flat plate	16	20						•			•
			RC framing	Post-tensioned flat plate	17	30									
		Reinforced concrete moment frame (perimeter)	RC framing	RC flat plate	18	10	•		•						
			RC framing	RC flat plate	19	20									
		RC, MF (perimeter) & SW (dual)	RC framing	RC flat plate	20	20									
			RC framing	RC flat plate	20A	30					•				
Office	Bearing walls	Reinforced concrete wall (core)	RC framing	RC flat slab	21	10									
			RC framing	RC flat slab	22	20				•	•				
		PC wall (interior & exterior)	PS framing	Prestressed slab	23	10									
	Complete Vertical Load Carrying Frame	Reinforced concrete shear wall	Steel framing	Steel beam & RC slab	24	10		•						•	
			Steel framing	Steel beam & RC slab	25	20									
		Steel braced frame	Steel framing	Steel beam & RC slab	26	20									
			Steel framing	Steel beam & RC slab	26A	5									•
		Steel moment frame	Steel framing	Steel beam & RC slab	27	10	•		•						
			Steel framing	Steel joist	M27A	5					•				
			Steel framing	Steel beam & RC slab	F27A	5							•		
			Steel framing	Steel beam & RC slab	28	40									
			Steel framing	Steel beam & RC slab	28A	30					•				
		Steel MF & RC SW (dual)	Steel framing	Steel beam & RC slab	29	20	•								
		Steel MF and BF (dual)	Steel framing	Steel beam and RC slab	30	20		•							
		Reinforced concrete moment frame	RC framing	Post-tensioned flat slab	31	10									
			RC framing	RC pan joist & waffle	32	10				•	•				
			RC framing	PT pan joist	33	20									
		RC, MF & SW (dual)	RC framing	RC pan joist & waffle	34	20	•								
Industrial	Bearing walls	Concrete masonry wall	Steel framing	Steel joist	35	L				•					
		PC wall (maybe PS)	PS framing	Prestressed slab	36	H									
			PS framing	PC double tees girders & beam	36A	H						•			
		PC tilt-up wall	Wood framing	Wood (plywood)	37	L	•								
	Complete Vertical Load Carrying Frame	PC tilt-up wall	Steel framing	Steel joist	38	L			•						
		Steel moment frame (transverse)/ braced frame (longitudinal)	Steel framing	Steel purlins & deck	39	L	•							•	
			Steel framing	Steel long-span truss	40	H		•							
		Concrete masonry wall	Steel framing	Steel joist	41A	2					•				
Commercial	Complete Vertical Load Carrying Frame	Concrete masonry wall	Steel framing	Steel joist	41	2	•								
			Steel framing	Steel joist (irregular plan form)	42	2			•						
		PS moment frame	PS framing	Prestressed slab	43	2									

1 - All office buildings will have a high first story, the industrial buildings are all on one story (with the exception of Building No. 41A) and for them the L indicates a low clearance, and H indicates a high clearance.

2 - BF = braced frame
PC = precast concrete
PS = prestressed, precast concrete
MF = moment frame
PT = post-tensioned concrete
RC = reinforced concrete
SW = shear wall (non-bearing)

3 - With the exception of the industrial building with purlins and steel deck (the metal building) all moment frames in Los Angeles, Seattle, and Memphis are to be Special. All moment frame in dual systems must also be Special. All other moment frames may be Ordinary.

TABLE C-2 Design Firms and Types of Building Designs

City/Design Firm	Building Type/Number from Table C-1
Charleston, South Carolina	
Enwright Associates, Inc.	5-story brick and RC block bearing walls (R)/CSC 6; 10-story steel frame with RC shear walls (O)/CSC 24; 1-story steel moment and braced frame (I)/CSC 39
Chicago, Illinois	
Alfred Benesch and Company	3-story brick and RC block bearing walls with plywood floor and roof diaphragms (R)/C 2A; 20-story RC frame with RC shear walls (R)/C 16
Klein and Hoffman, Inc.	12-story RC bearing wall (R)/C 9; parametric study of steel moment and/or braced frames (O)/C 26, 27, 30; 1-story precast RC bearing walls with PC double tee roof (I)/C 36A
Ft. Worth, Texas	
The Datum/Moore Partnership	5-story RC block walls with prestressed slabs (R)/FW 3; 10-story RC frame with RC shear walls (R)/FW 15; 5-story steel moment frame (O)/FW 27A
Los Angeles, California	
S. B. Barnes and Associates	3-story wood with plywood walls (R)/LA 1; 1-story wood frame with precast concrete tilt-up walls (I)/LA 37; 1-story steel with moment and braced frames (I)/LA 39; 2-story steel frame with RC block walls (C)/LA 41
Johnson and Nielsen Associates	20-story steel moment frame with shear walls (dual) (O)/LA 3
Wheeler and Gray	12-story reinforced brick bearing wall with RC slabs (R)/LA 5; 10-story RC frame with shear walls (R)/LA 15; 10-story RC frame (perimeter) with RC slabs (R)/LA 18; 10-story steel moment frame LA 27(O)/
Memphis, Tennessee	
Allen and Hoshall	5-story bearing wall (R)/M 8; 1-story steel frame with RC tilt-up exterior shear walls (I)/M 38; 2-story steel frame with nonbearing RC block walls (C)/M 42
Ellers, Oakley, Chester and Rike, Inc.	20-story steel moment and braced frame with RC slab floors (R)/M 14; 10-story RC moment frame (perimeter) (R)/M 18; 10-story steel moment frame (special) with RC slabs (O)/M 27

TABLE C-2 continued

City/Design Firm	Building Type/Number from Table C-1
New York, New York	
Weidlinger Associates	12-story brick bearing wall (R)/NY 5; 30-story RC moment frame and nonbearing shear wall (dual) (R)/NY 20A; 10-story RC moment frame (O)/NY 32
Robertson, Fowler and Associates PC	20-story RC bearing wall (O)/NY 22; 5-story steel moment frame (O)/NY 27A; 30-story steel moment frame (O)/NY 28A; 2-story steel frame with RC block walls (I)/NY 41A
Phoenix, Arizona	
Magadini-Alagia Associates	5-story RC bearing wall (R)/P 10; 20-story RC bearing wall with core shear walls (O)/P 22; 10-story RC frame (ordinary) (O)/P 32
Read Jones Christoffersen, Inc.	3-story RC block bearing wall (R)/P 2; 5-story RC block bearing wall (R)/P 3; 1-story steel frame with RC block shear walls (I)/P 35
St. Louis, Missouri	
Theiss Engineers, Inc.	10-story clay brick bearing wall (R)/SL 5A; 20-story RC frame with shear walls (R)/SL 16; 5-story steel frame with braced frames at core (O)/SL 26A
Seattle, Washington	
ABAM Engineers, Inc.	10-story steel frame with RC shear walls (O)/S 24
Bruce C. Olsen	3-story wood with plywood walls (R)/S 1; 1-story long-span steel, 30-foot clear height, moment and braced frames (I)/S 40
Skilling Ward Rogers Barkshire, Inc.	20-story steel frame-dual special and braced frames (O)/S 30

consultant, reviewed the design concept and approach at various stages (interpretation of design criteria, analysis of load effects, and completion of design). The design firms were asked to certify the accuracy of their calculations and to report their findings. During Phase II of the program, attention also was given to the regulatory aspects of the amended *Tentative Provisions*.

For each of the 52 designs included, a set of building requirements or general specifications was developed and provided to the responsible design engineering firm, but the designers were given latitude to ensure that building design parameters such as bay size were compatible with local construction practice. The designers were not permitted, however, to change the basic structural type. For example, they could not change from a reinforced concrete frame system specified in the building requirements to a reinforced concrete shear wall system. Such changes were not permitted even if an alternative structural type would have cost less than the specified type under the early version of the *Provisions*, and this constraint may have prevented the designer from selecting the most economical system. Consequently, some of the cost impact information discussed in the "Chapter 1 Commentary" may reflect overly high estimates for some trial designs.

Phase II concluded with publication of:

- A draft version of the recommended provisions, *The NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, that included three parts--the draft provisions, the draft commentary to the provisions, and an appendix that presented Chapters 13-15 of the original ATC document concerning existing buildings;

- An overview of Phases I and II of the BSSC program that included the BSSC-NBS Overview Committee's analysis of the results and the executive summaries from the reports of the design firms participating in the program as well as a series of appendixes that presented the initial amendments to the original ATC document, the original trial design program plan, the plan for studies to be conducted in Phase III of the program, the detailed contract work plans for Phases I and II, and a list of the members of the BSSC technical committees.

- The design firms' reports.

DEVELOPMENT OF THE 1985 EDITION OF THE *NEHRP RECOMMENDED PROVISIONS*

The draft provisions issued at the conclusion of Phase II reflected the initial amendments to the original ATC document as well as further refinements made by the Overview Committee during Phases I and II of the program. They represented an interim set of provisions pending their balloting by the BSSC member organizations during Phase III of the BSSC program, which began in July 1984.

The first ballot, which was conducted in accordance with the BSSC Charter, was organized on a chapter-by-chapter basis using a form that provided for four responses: yes, yes with reservations, no, and abstain. All "yes with reservations" and "no" votes were to be accompanied by an explanation of the reasons for the vote and the "no" votes were to be accompanied by specific suggestions for change if those changes would change the negative vote to an affirmative.

All comments and "yes with reservation" and "no" votes received as a result of the first ballot were compiled. Proposals for dealing with these responses then were developed for consideration by the Technical Overview Committee and, subsequently, the BSSC Board of Direction. The draft provisions were then revised to reflect the changes deemed appropriate by the BSSC Board and were submitted to the BSSC membership for balloting again in August-September 1985.

As a result of this second ballot, virtually the entire provisions document received consensus approval, and a special BSSC Council meeting was held in November 1985 to resolve as many of the remaining differences as possible. The 1985 Edition of the *NEHRP Recommended Provisions* then was transmitted to FEMA for publication in December 1985.

COMPLEMENTARY DOCUMENTS

During the next three years, a number of documents were published to support and complement the 1985 Edition of the *NEHRP Recommended Provisions* and a project was initiated to stimulate widespread use of the *Provisions* (Phase V of the BSSC Program). The reports issued include the following:

- A guide to application of the *Provisions* in earthquake-resistant building design.
- A nontechnical explanation of the *Provisions* for the lay reader.
- A handbook for interested members of the building community and others explaining the societal implications of utilizing improved seismic safety provisions and a companion volume of selected readings.
- Handbooks identifying seismic design considerations for the owners and other decisionmakers and design professionals responsible for apartment buildings, elementary and secondary

schools, health care facilities, hotels and motels, and office buildings.

THE UPDATE EFFORT TO PRODUCE THE 1988 EDITION

The need for continuing revision of the *Provisions* had been anticipated since the onset of the BSSC program and the effort (Phase IV of the BSSC Program) to update the 1985 Edition for reissuance in 1988 began in January 1986. During the update effort, nine BSSC Technical Committees were formed to focus on seismic risk maps, structural design, foundations, concrete, masonry, steel, wood, architectural/mechanical/electrical systems, and regulatory use. The Technical Committees (TCs) worked under the general direction of a Technical Management Committee (TMC), whose function was similar to that of the Overview Committee used during earlier phases of the program. (The participants in the Phase IV update effort are identified in Appendix B.).

The TMC was composed of a representative of each TC as well as additional members identified by the Board to provide balance. It served as the effort coordinator and was charged to deal with global issues; to provide the continuing liaison between the TCs and the BSSC Board of Direction, which holds ultimate responsibility for all BSSC programs; to consider and respond to all comments and negative votes received as a result of the balloting for the 1988 Edition; and to prepare recommendations for resolving issues raised as a result of the balloting.

The TCs were composed of individuals nominated by organizations deemed by the BSSC Board to have both an interest and expertise in the various subjects to be addressed. When additional technical expertise was deemed necessary, the Board made additional appointments. Basically, the TCs were charged to considered new developments (e.g., newly issued standards) and experience data that had become available (e.g., as a result of the 1985 Mexico City earthquake) since issuance of the 1985 Edition of the *Provisions* as well as those issues left unresolved at the conclusion of the Phase III effort.

The unresolved issues, which numbered 58, focused on the risk maps; the Seismicity Index and Seismic Performance Categories; R factors (inelastic reduction factors); strength versus working stress design; drift limits; C_T factors (approximate periods of vibration); P-delta limits (gross stability); modal analysis procedures, soil-structure interaction; foundation design requirements; and various issues in the chapters on architectural, mechanical and electrical components and systems, wood, steel, concrete, and masonry. Each unresolved issue was addressed by at least one TC; some were submitted as proposals for change for the 1988 Edition, some were incorporated as minor editorial revisions, some were considered and rejected at the TC level, and some were deferred for study in future update efforts due to the lack of available data or time.

A number of new issues also were raised for consideration during the update effort and some were more philosophical than technical. It was deemed appropriate, for example, to have all the TCs and the TMC reassess the intent of the *Provisions* as stated in the opening paragraphs of the 1985 Edition, and some committees also discussed whether damage control in areas of low seismicity should be considered in the *Provisions* in addition to life safety. As a result of these deliberations, several revisions were proposed to clarify the overall objectives of the document. Another cluster of new issues concerned the relationship of the *NEHRP Recommended Provisions* to other structural and seismic provisions. The idea of working towards a common format to ease incorporation of one body's standards into another's was endorsed. Several proposals also were made to bring the *Provisions* into conformance with the new editions of the *Uniform Building Code* and Structural Engineers Association of California's *Blue Book*. These proposals did not simply involve direct adoptions; rather, they recognized the importance and validity of the research behind the changes in the other documents. Other new standards such as the ACI-ASCE 530-88 masonry code and the LRFD specification for steel design being developed by the American Institute of Steel Construction also stimulated proposals for change.

The TCs and TMC worked throughout 1987 to develop specific proposals for changes needed in the 1985 Edition of the *Provisions*. In December 1987, the Board reviewed specific proposals for change that had been developed by the TCs and TMC and decided upon a set of 53 proposed revisions to the 1985 Edition of the *Provisions* for submittal to the BSSC membership for ballot. Approximately half of the proposals reflected new issues while the other half reflected efforts to deal with the unresolved 1985 issues.

The ballot was mailed to each BSSC member organization in February 1988 for submittal in April. The ballot was conducted on an proposal-by-proposal basis using a form that provided for four responses: yes, yes with reservations, no, and abstain.[1] Fifty of the proposal items on the ballot passed and three failed. All comments and "yes with reservation" and "no" votes received as a result of the ballot were compiled for review by the TMC. Many of the comments could be addressed by making minor editorial adjustments and these were approved by the Board. Other comments were found to be unpersuasive or in need of further study during the next update cycle and, consequently, no changes were made in response to these comments. Finally, a number of

[1]As was the case in the balloting of the 1985 Edition, the "yes with reservations" and "no" votes were to be accompanied by an explanation of the reasons for the vote and the "no" votes were to be accompanied by specific suggestions for change if those changes would change the negative vote to an affirmative.

comments persuaded the TMC and Board that a substantial alteration of a balloted proposal was necessary, and it was decided to submit these matters (11 in all) to the BSSC membership for reballot. The reballoting began in June 1988 and concluded in July. Nine of the eleven reballot proposals passed and two (concerned with increasing the R and C_d factors for ordinary and intermediate reinforced concrete frames) failed.

On the basis of the ballot and reballot results, the 1988 Edition of the *Provisions* was prepared and transmitted to FEMA for publication in August 1988. A report describing the changes made in the 1985 Edition and issues in need of attention in the next update cycle then was prepared and efforts began to revise the complementary reports published to support the 1985 Edition.

BUILDING SEISMIC SAFETY COUNCIL MEMBERS

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* Affiliate Members