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1994 Edition

NEHRP RECOMMENDED PROVISIONS FOR SEISMIC REGULATIONS FOR NEW BUILDINGS

Part 2 - Commentary

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BUILDING
SEISMIC
SAFETY
COUNCIL

Program
on
Improved
Seismic
Safety
Provisions

A council of the National Institute of Building Sciences

1994 Edition

NEHRP RECOMMENDED PROVISIONS FOR SEISMIC REGULATIONS FOR NEW BUILDINGS

Part 2

Commentary

THE BUILDING SEISMIC SAFETY COUNCIL AND ITS PURPOSE

The Building Seismic Safety Council (BSSC) was established in 1979 under the auspices of the National Institute of Building Sciences (NIBS) as an entirely new type of instrument for dealing with the complex regulatory, 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 issues related to the seismic safety of the built environment could be resolved and jurisdictional problems overcome through authoritative guidance and assistance backed by a broad consensus.

The BSSC 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.

See Appendix E of the *Commentary* volume for a full description of the BSSC program.

BSSC Program on Improved Seismic Safety Provisions

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

1994 EDITION

Part 2: COMMENTARY

Prepared by the
Building Seismic Safety Council
for the
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BUILDING SEISMIC SAFETY COUNCIL
Washington, D.C.
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This report was prepared under Contract EMW-C-0903 between the Federal Emergency Management Agency and the National Institute of Building Sciences.

Building Seismic Safety Council reports include the documents listed below; unless otherwise noted, single copies are available at no charge from the Council:

NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, 2 volumes and maps, 1994

NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings, 1991 Edition, 2 volumes and maps, 1991

Guide to Use of the 1991 NEHRP Recommended Provisions in Earthquake-Resistant Design of Buildings, 1995

Non-Technical Explanation of the NEHRP Recommended Provisions, Revised Edition, 1995

Seismic Considerations for Communities at Risk, 1995

Seismic Considerations: Elementary and Secondary Schools, Revised Edition 1990

Seismic Considerations: Health Care Facilities, Revised Edition, 1990

Seismic Considerations: Hotels and Motels, Revised Edition, 1990

Seismic Considerations: Apartment Buildings, 1995

Seismic Considerations: Office Buildings, 1995

Societal Implications: Selected Readings, 1986.

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

Strategies and Approaches for Implementing a Comprehensive Program to Mitigate the Risk to Lifelines from Earthquakes and Other Natural Hazards, 1989 (available from the National Institute of Building Sciences for $11)

For further information concerning any of these documents or the activities of the BSSC, contact the Executive Director, Building Seismic Safety Council, 1201 L St., N.W., Suite 400, Washington, D.C. 20005.

CONTENTS

NEHRP RECOMMENDED PROVISIONS FOR SEISMIC REGULATIONS FOR NEW BUILDINGS

1994 EDITION

Part 2: COMMENTARY

NOTE

Those portions of the 1994 Edition of this *Commentary* volume that are substantively different from the 1991 Edition are identified in the margins as follows:

Additions
or
revisions

Deletions

Not highlighted are editorial and format changes; however, Appendix A of the the *Provisions* volume presents a summary of the substantive differences between the 1991 and 1994 Editions and includes an explanation of the format changes and a comparison of 1991 and 1994 chapter and section numbers.

Chapter 1 Commentary

GENERAL PROVISIONS

Chapter 1 sets forth general requirements for applying the analysis and design provisions contained in Chapters 2 through 9 of the *NEHRP Recommended Provisions*. It is similar to what might be incorporated in a code as administrative regulations.

Chapter 1 is designed to be as compatible as possible with normal code administrative provisions (especially as exemplified by the three national model codes), but 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, although the *NEHRP Recommended Provisions* is national in scope, it presents minimum criteria. It is neither 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, "Alternative 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, may be a law-making body such as a state legislature, a city council, or some other 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 for 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.

Note that appendices to this *Commentary* volume present general discussions of topics related to the *Provisions*. Appendix A describes the development of *Provisions* Maps 1 through 4; Appendix B discusses the cost implications of application of the *Provisions*; Appendix C provides a discussion of risk; and Appendix D presents additional background concerning the site response provisions introduced in the 1994 *Provisions*.

1.1 PURPOSE: The goal of the *NEHRP Recommended Provisions* is to present criteria for the design and construction of new 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 after an earthquake. To this end, the *Provisions* provides 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* document has been reviewed extensively and balloted by the building community and, therefore, it is 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, 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 the *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 for most buildings. The objective of the *Provisions* therefore is 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), the *Provisions* presents only minimum criteria in general terms. These criteria reflect both scientific and engineering data supplemented with judgment based on past earthquake experiences and design applications and, as a result, significant variability is associated with them.

The process of designing and constructing an earthquake-resistant structure involves several steps that require use of input parameters with significant variability.

Table C1.1 lists the key steps involved in the seismic design process and estimates of the variability associated with each step along with a more detailed breakdown of the parameters associated with each step. The ranges of variability are intended to give some guidance to the user of the *Provisions* by pointing out the possible range of values provided, calculated, or used in the seismic design-construction process. They are not related to code values. They are a judgmental quantification developed by the 1994 BSSC Design Values Panel after extensive discussion and review. They were not computed from statistics and, thus, should not be used to impute levels of reliability.

TABLE C1.1
Major Seismic Design Input Parameters

Design Step	Parameters	Approximate Variability
Seismic Hazard	Source zones Recurrence rates Maximum magnitudes Ground motion attenuation	100% ±
Site Response Factors	Soil/rock impedance Soil depths Soil properties Strain levels	40% ±
Structural Analysis	Material properties Damping Elastic moduli Weights Modeling of structure Modeling of foundation Combination of modal responses Combination of directions *R* factors/ductility	0 to 20% 0 to 50% 0 to 50% 15% ± 15% ± 0 to 50% 0 to 50% 0 to 50% 0 to 50%
Structural Design	Seismic Hazard Exposure Group Selection of seismic resistant system Redundancy Story drift determination Detailing requirements *P*-delta effects Load combinations Material strength properties	40% ±
Quality Assurance and Control Procedures	Application of the procedures Review of the calculations and plans Material tests	100% ±
Construction Practices	Use of proper materials Proper placement and erection of materials and components Inspection	25% ±

The goal of the seismic design process is to design a structure that will remain stable when it is subjected to seismic forces. Thus, the structure should be designed with appropriate margins or conservatism considering the variability of the input. The design steps associated with determining seismic demand are the seismic hazard, the site response factors, and the structural analysis. The design steps associated with seismic capacity are structural design data, quality assurance and control procedures, and construction practices.

The seismic demand parameter with possibly the greatest associated variability involves seismic hazard determination and, consequently, use of the seismic hazard maps. Most of this uncertainty derives from the nature of seismicity. Due to uncertainty in ground motion attenuation functions, the U.S. Geological Survey (USGS) has performed sensitivity studies for several U.S. cities to quantify the variability associated with the attenuation functions used in the development of the spectral response seismic hazard maps in the 1994 *Provisions* Maps 5 through 12. These studies suggest that the seismic hazard values in the *Provisions* range from 30 to 70 percent above the values obtained when the median attenuation functions are used without considering the uncertainty associated with the attenuation functions except in some areas with higher seismic hazard. Also, Thenhaus and coworkers. (1987) have demonstrated that the variability in expected ground motion in the eastern United States due to different characterizations of seismic source zones is 1.2 to 3.1 (ratio between the maximum and minimum ground motions). Similar or greater variability exists with respect to the seismic hazard maps defining effective peak acceleration (EPA) and effective peak velocity (EPV) in the *Provisions*, Maps 1 through 4. The EPA and EPV values are used with standard spectral shapes which, for many U.S. sites, provide conservatism especially at fundamental periods over about 1 sec. This conservatism results because the standard spectral shapes were developed to envelope all site conditions and earthquake magnitudes.

Site response factors are another seismic demand parameter with significant variability. The factors in the 1994 *Provisions* were developed as a result of discussions and recommendations made at a National Center for Earthquake Engineering Research/Structural Engineers Association of California/Building Seismic Safety Council Site Response Workshop in November 1992. Following the workshop it was suggested that the acceleration based site-coefficient (F_a) values generally represent mean values and the velocity-based site coefficient (F_v) values represent site response factors between the mean and the mean plus about one sigma (standard deviation).

The last step in defining seismic demand is determination of the earthquake structural forces by analyses. As shown in Table C1.1, the structural damping, elastic moduli, weights, modeling (including soil-structure interaction), combination of modes, combination of directions, and the R factors (representing inelastic behavior and other energy dissipation characteristics) all contribute to the variability in determining the structural forces. Note that although the R factor could be considered to be part of seismic capacity instead of demand, it is considered on the demand side of the equation because it is used to determine the structural loads. All of these parameters affect prediction of the seismic structural responses. The estimates in Table C1.1 represent the best judgment of the BSSC Design Values Panel members participating in the 1994 *Provisions* update effort and represents their opinions based on nearly 300 years of cumulative structural design experience. The ASCE Working Group on Quantification of Uncertainties of the Committee on Dynamic Analysis of the Committee on Nuclear Structures (1986) discussed in some detail the variability in most of these parameters and generally supported the variability of 0 to 50 percent in soil-structure interaction. As the ASCE Working Group indicated, the combined effect of all these variabilities is desirable, but it is difficult to determine. Ang and Newmark (1977) have performed some simplified studies that suggest the total variability with respect to the prediction of structural response could be from 0 to 60 percent.

The design steps associated with determining the seismic capacity are the structural design process, quality assurance and control (QA&C) procedures, and construction practices. The various parameters of the structural design process identified in Table C1.1 could be generalized in two parts--structural design and material strength properties. The variability in structural design (story drift, detailing requirements, *P*-delta effects, load combinations) was identified as 10 to 25 percent. The variability in material strength properties was identified as 15 to 20 percent, which is similar to values reported by the ASCE Working Group (1986). Table C1.1 reports a combined value for structural design.

The variability associated with construction practices was estimated to be about 10 to 25 percent while the variability associated with QA&C procedures could be from 0 to 100 percent depending on whether QA&C procedures are prepared and implemented. The main point to be made is that QA&C procedures along with construction procedures and inspection are essential to ensure that a structure is built to satisfy the seismic design requirements. Proper implementation of the procedures should limit the variability such that the seismic safety of the structure is not impacted.

As stated above, the goal of the design process is to ensure that the seismic capacity of the structure exceeds the seismic demand placed on the structure with appropriate margins or conservatism considering the variabilities of the input. It would be desirable to quantify the appropriate margins and conservatism but it is difficult to do so. However, for the present, the experienced structural engineer together with the design team must be relied upon to exercise judgment in interpreting and adapting the basic principles and the relevant associated variabilities to a specific project.

The *Provisions* document is 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 acceleration and velocity coefficients with 90 percent probabilities of not being exceeded in 50 years and another set giving acceleration and velocity coefficients 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 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 these elements are tied rigidly to the structure, they 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 the *Provisions*.

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

A 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:

1. To ensure that the design has fully identified the seismic force resisting system and its appropriate design level and

2. To ensure that the design basis is 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 exempt and need not comply:

1. Detached one- and two-family dwellings located at sites where the seismic coefficient C_a is less than 0.15 also are exempt because they represent exceptionally low risks (see Sec. 1.2).

2. A simple procedure is specified for detached 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. 9.10 are adequate to provide the safety required based on the history of such frame construction--especially low structures--in earthquakes.

3. Agricultural storage buildings are generally exempt from most code requirements because of the exceptionally low risk to life involved and that is the case of the *Provisions*.

Existing buildings, except for additions thereto and changes of occupancy therein, are not within the scope of the *Provisions*. FEMA, however, currently is sponsoring a program on mitigation of the seismic hazard to existing buildings; for information, write FEMA, Earthquake Programs, Washington, D.C. 20472. For the first time, this 1994 Edition of the *Provisions* includes a section (Sec. 2.7) that presents guidance for the seismic design of nonbuilding structures such as vessels, silos, chimneys, and towers. Other structures such as power plants, bridges, dams, retaining walls, docks, and off-shore platforms require seismic design procedures that are beyond the scope of the *Provisions* and the structural engineer must establish criteria to suit the special requirements for performance and reliability of these structures.

The *Provisions* are not written to prevent damage due to earth slides (such as those that occurred in Anchorage, Alaska), to liquefaction (such as occurred in Niigata, Japan), or to tsunami (such as occurred in Hilo, Hawaii). It provides 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 9 to new buildings, additions to existing buildings, and changes of use are established in this section.

1.3.2 ADDITIONS TO EXISTING BUILDINGS: The requirements for additions--both horizontal and vertical reflect the fact that the *Provisions* does not include criteria for alterations and repairs to existing buildings. Thus, additions that are structurally independent of an existing building are considered to be new buildings required to conform with the *Provisions.* For additions that are not structurally independent, the intent is that the addition as well as the existing building be made to comply with the *Provisions* except that an increase of up to 5 percent of the mass contributing to seismic forces is permitted in any elements of the existing building without bringing the entire building into conformance with the *Provisions.*

1.3.3 CHANGE OF USE: Although the *Provisions* document does 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 (SHEG), the building must be made to conform to the *Provisions* for the new Seismic Hazard Exposure Group except that buildings in areas with an effective peak velocity-related acceleration (A_v) value of less than 0.15 being reclassified from SHEG I to SHEG II need not comply.

1.4 SEISMIC PERFORMANCE: The *Provisions* requirements for analysis and design of buildings 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 the hazard based on a measure of risk.

The purpose of Sec. 1.4.1 and 1.4.2 is to provide the means for establishing the 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* is identified.

1.4.1 SEISMIC GROUND ACCELERATION MAPS: This section introduces the seismic ground acceleration maps, Maps 1 through 4 and the values of effective peak acceleration (A_a) and effective peak velocity-related acceleration (A_v) for the various map areas. See Appendix

A of this *Commentary* volume for a discussion of the development of the maps. Probabilistically based Maps 1 through 4 were developed about 20 years ago, and newer data (see the "Appendix of Chapter 1" of the *Provisions*) indicate that these maps could underestimate the seismic exposure in some areas of the country, especially portions of the Pacific Northwest, California, Utah, New England, and the Mississippi Valley. Jurisdictions in those areas and others using these guidelines for those areas should consider all available information before selecting A_a and A_v values for their jurisdiction or use.

In Sec. 1.4.1.2, it is intended that the values of A_a and A_v may be determined directly from Maps 3 and 4 instead of the A_a and A_v values in Table 1.4.1.1. Maps 3 and 4 are contour maps of effective peak acceleration coefficients. For a particular site or jurisdiction, the A_a and A_v values may be read by interpolation of the contour maps or the highest A_a and A_v value may be selected for design purposes.

Local jurisdictions may wish to stipulate a single A_a and A_v value for design which would eliminate potential interpolation interpretations. The selection of a single value for jurisdiction use would be extremely conservative for large geographical areas where the A_a and A_v values vary significantly within the boundaries of the jurisdiction. For this situation, a jurisdiction may elect to allow direct interpolation of Maps 3 and 4 for a given site within the jurisdiction boundaries.

1.4.2 SEISMIC COEFFICIENTS: This commentary section focuses on site coefficients and related topics. Sec. 2.5 and its commentary provide background on soil-structure interaction. Sec. 1.4.5 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.

In previous editions of the *Provisions*, the term A_v was used as a "trigger" value throughout. The A_v term was chosen because it was equal to or greater than A_a in all map areas. The 1994 Edition of the *Provisions* introduces new site coefficients, F_a and F_v, that recognize the nonlinearity of the soils factors, and it was concluded that most trigger values should be modified to incorporate these factors. However, the terms A_aF_a and A_vF_v were perceived as being cumbersome and it therefore was decided to introduce the new C_a and C_v terms which are based on the following formulas:

$$C_a = A_aF_a \text{ and } C_v = A_vF_v$$

Unfortunately, this decision was reached relatively late in the 1994 update process and there was insufficient time to thoroughly explore the impact of making this change in all places where the A_v trigger appeared, most notably in Table 1.4.4. It should be clearly noted, however, that the intent of the 1994 PUC is that future editions of the *Provisions* will have the site coefficients incorporated into Table 1.4.4 as well as in other sections where the A_v trigger was retained for 1994.

In general, the A_v trigger value has been replaced with the seismic coefficient C_a in the 1994 *Provisions*. One notable exception is Table 2.3.3 where C_v is used as the trigger value. Since the purpose of the table is to determine the upper bound limit for calculating the fundamental period, it was felt that maintaining a velocity-related coefficient was more appropriate in this instance.

The values for the site coefficients, F_a and F_v, used to determine the seismic coefficients, C_a and C_v, are shown in Tables 1.4.2.3a and 1.4.2.3b but are not used directly in the *Provisions*. *Provisions* Tables 1.4.2.4a and 1.4.2.4b were developed by multiplying the values in Tables 1.4.2.3a and 1.4.2.3b for each of the Soil Profile Types by the acceleration coefficients, A_a and A_v, respectively.

Site Conditions: It has long been recognized that the effects of local soil conditions on ground motion characteristics should be considered in building design, and most countries considering these effects have developed different design criteria for several different soil conditions. The 1989 Loma Prieta earthquake provided abundant strong motion data that was used extensively together with other information in developing the 1994 *Provisions*. Evidence of the effects of local soil conditions has been observed globally including eastern North America. An example of the latter is a pocket of high intensity reported on soft soils in Shawinigan, Quebec, approximately 155 miles (250 km) from the 1925 Charlevoix magnitude 7 earthquake (Milne and Davenport, 1969).

The Applied Technology Council (ATC) study that generated the preliminary version of the *Provisions* provided for the use of three Soil Profile Types considered, in the late 1970s, to be different enough in seismic response to warrant separate site coefficients (S factors) and experience from the September 1985 Mexico City earthquake prompted the addition of a fourth Soil Profile Type. These have been revised for the 1994 *Provisions* to conform to the experiences of the Mexico City and the 1989 Loma Prieta earthquake in California as well as to other observations and studies showing the effects of level of shaking, rock stiffness, and soil type, stiffness and depth on the amplification of ground motions at short and long periods. The resulting use of higher seismic coefficients in areas of lower shaking and the addition of a "hard rock" category in the 1994 *Provisions* better reflect the conditions in some parts of the country and incorporate recent efforts toward a seismic code for New York City (Jacob, 1990 and 1991). The need for improvement in codifying site effects was discussed at a 1991 National Center for Earthquake Engineering Research (NCEER) workshop devoted to the subject (Whitman, 1992), which made several general recommendations. At the urging of Robert V. Whitman, a committee was formed during that workshop to pursue resolution of pending issues and develop specific code recommendations. Serving on this committee were M. S. Power (chairman), R. D. Borcherdt, C. B. Crouse, R. Dobry, I. M. Idriss, W. B. Joyner, G. R. Martin, E. E. Rinne, and R. B. Seed. The committee collected information, guided related research, discussed the issues, and organized a November 1992 Site Response Workshop in Los Angeles (Martin, 1994). This workshop discussed the results of a number of empirical and analytical studies and approved consensus recommendations that form the basis for the 1994 *Provisions*.

Amplification of Peak Ground Acceleration: Seed and coworkers (1976a) conducted a statistical study of peak accelerations developed at locations with different site conditions using 147 records from each western U.S. earthquake of about magnitude 6.5. Based on these results, judgment and analysis, they proposed the acceleration relations of Figure C1.4.2-1a that are applicable to any earthquake magnitude of engineering interest. It must be noted that the data base of that study did not include any soft clay sites and, thus, the corresponding curve in the figure was based on the authors' experience and, consequently, was somewhat more speculative.

Idriss (1990a and 1990b), using data from the 1985 Mexico City and 1989 Loma Prieta earthquakes, recently modified the curve for soft soil sites as shown in Figure C1.4.2-1b. In these earthquakes, low maximum rock accelerations of 0.05g to 0.10g were amplified by factors of from about 1.5 to 4 at sites containing soft clay layers ranging in thickness from a few feet to more than a hundred feet and having depths of rock up to several hundred feet. As shown by the data and site response calculations included in Figure C1.4.2-1b, the average amplification factor for soft soil sites tends to decrease as the rock acceleration increases--from 2.5 to 3 at low accelerations to about 1.0 for a rock acceleration of 0.4g. Since this effect is directly related to the nonlinear stress-strain behavior in the soil as the acceleration increases, the curve in Figure C1.4.2-1b can be applied in first approximation to any earthquake magnitude of engineering interest.

It is clear from Figure C1.4.2-1b that low peak accelerations can be amplified several times at soil sites, especially those containing soft layers and where the rock is not very deep. On the other hand, larger peak accelerations can be amplified to a lesser degree and can even be slightly deamplified at very high rock accelerations. In addition to peak rock acceleration, a number of factors including soil softness and layering play a role in the degree of amplification. One important factor is the impedance contrast between soil and underlying rock.

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; Hayashi et al., 1971).

The mean spectral shapes determined directly from the study by Seed and coworkers (1976b), based on 104 records from 21 earthquakes in the western part of the United States, Japan and Turkey, are shown in Figure C1.4.2-2. The ranges of magnitudes and peak accelerations covered by this data base are 5.0 to 7.8 and 0.04*g* to 0.43*g*, respectively. All spectra used to generate the mean curve for soft to medium clay and sand in Figure C1.4.2-2 correspond to rather low peak accelerations in the soil (less than 0.10*g*). The spectral shapes in the figure 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 C1.4.2-3. The curves in this figure therefore apply to the three soil conditions in the original version (1985) of the *Provisions.*

The three conditions corresponding to the three lines in Figure C1.4.3-3 plus a fourth condition introduced following the 1985 Mexico City earthquake are described as follows:

1. Soil Profile Type S_1--A soil profile with either: (1) rock of any characteristic, either shale-like or crystalline in nature, that has a shear wave velocity greater than 2,500 ft/s (762 m/s) or (2) stiff soil conditions where the soil depth is less than 200 ft (61 m) and the soil types overlying the rock are stable deposits of sands, gravels, or stiff clays.

2. Soil Profile Type S_2--A soil profile with deep cohesionless or stiff clay conditions where the soil depth exceeds 200 ft (61 m) and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

3. Soil Profile Type S_3--A soil profile containing 20 to 40 ft (6 to 12 m) in thickness of soft- to medium-stiff clays with or without intervening layers of cohesionless soils.

4. Soil Profile Type S_4--A soil profile characterized by a shear wave velocity of less than 500 ft/sec (152 m/s) containing more than 40 ft (12 m) of soft clays or silts.

The post-Loma Prieta studies (Martin, 1994) have resulted in considerable modification of these profile types resulting in the Soil Profile Types in the 1994 *Provisions*, A through F.

Response of Soft Sites to Low Rock Accelerations: Earthquake records on soft to medium clay sites subjected to low acceleration levels indicate that the soil/rock amplification factors for long-period spectral accelerations can be significantly larger than those in Figures C1.4.2-1 and C1.4.2-2 (Seed et al., 1974). Furthermore, the largest amplification often occurs at the natural period of the soil deposit. In Mexico City in 1985, the maximum rock acceleration was amplified four times by a soft clay deposit that would have been classified as S_4 whereas the spectral amplitudes were about 15 to 20 times larger than on rock at a period near 2 sec. In other parts of the valley where the clay is thicker, the spectral amplitudes at periods ranging between 3 and 4 sec also were amplified about 15 times, but the damage was less due to the low rock motion intensity at these very long periods (Seed et al., 1988). Inspection of the records obtained at some soft clay sites during the 1989 Loma Prieta earthquake indicates a maximum amplification of long-period spectral amplitudes of the order of three to six times. Figure C1.4.2-4 shows a comparison of average response spectra measured on rock and soft soil sites in San Francisco and Oakland during this magnitude 7.1 earthquake. A preliminary study of the Loma Prieta records at one 285-ft (87 m) soil deposit on rock containing a 55-ft (17 m) soft to medium stiff clay layer (Treasure Island) seems to suggest that the largest soil/rock amplification of response spectra occurred at the natural period of the soil deposit, similarly to Mexico City (Seed et al., 1990).

Some relevant theoretical and experimental findings are reviewed briefly below to clarify the role of key site parameters in determining the magnitude of the soil/rock amplification of spectral ordinates at long periods for sites containing soft layers. These parameters are the thickness of the soft soil, the shear wave velocity of the soft soil, the soil/rock impedance ratio (*IR*), the layering and properties of the stiffer soil between soft layer and rock, and the modulus and damping properties of the soft soil. The basic assumptions used are those typically used in one-dimensional site response analyses and, thus, the conclusions drawn are restricted to sites where these conditions are fulfilled (i.e., flat sites with horizontal layering of significant extension and far from rock outcrops and with a clear soil-rock interface at a depth not exceeding several hundred feet).

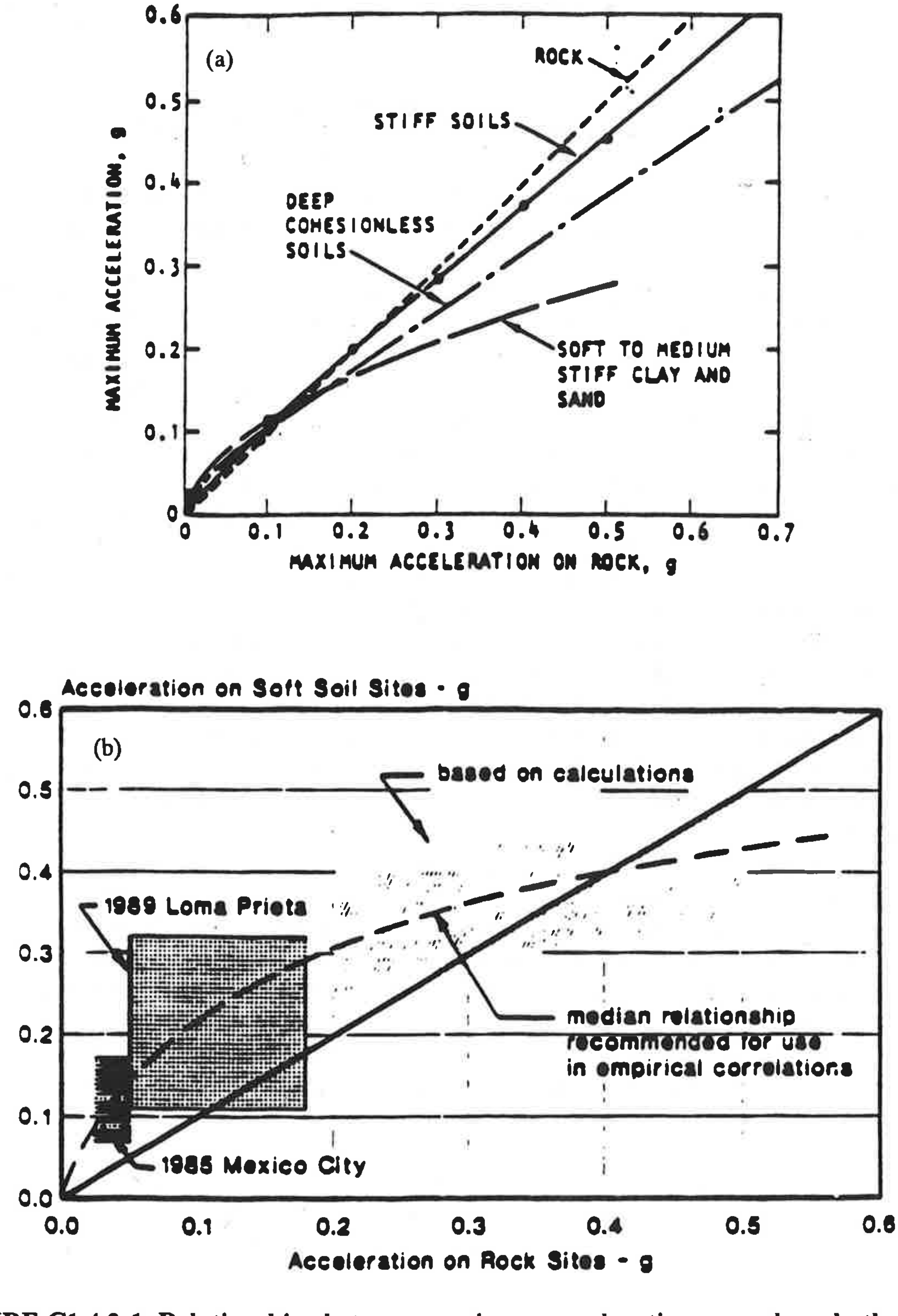


FIGURE C1.4.2-1 Relationships between maximum acceleration on rock and other local site conditions: (a) Seed et al., 1976a, and (b) Idriss, 1990a and 1990b.

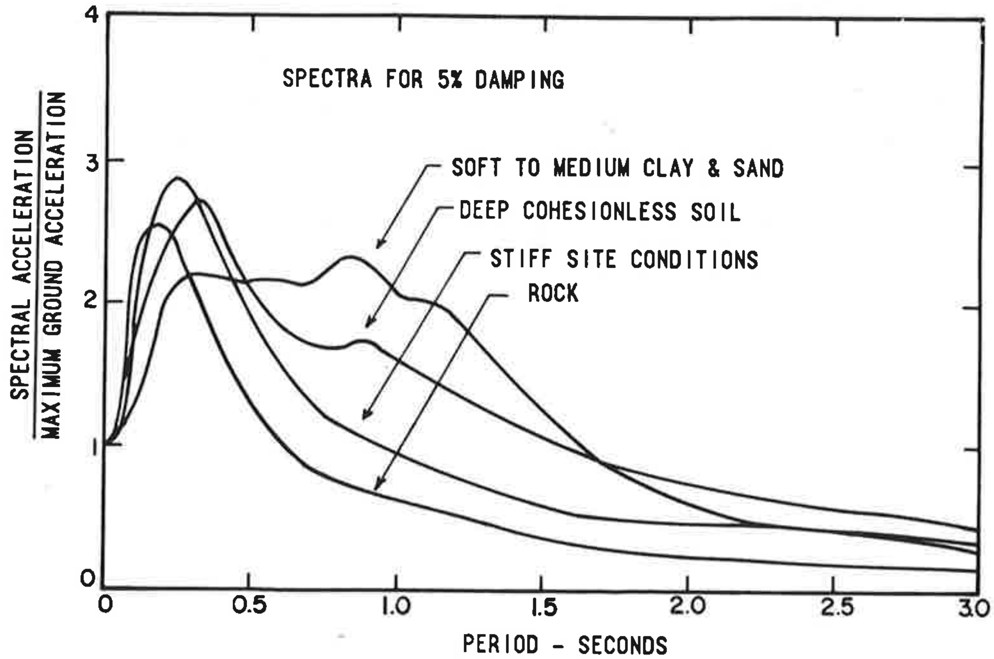


FIGURE C1.4.2-2 Average acceleration spectra for different site conditions (Seed et al., 1976a and 1976b).

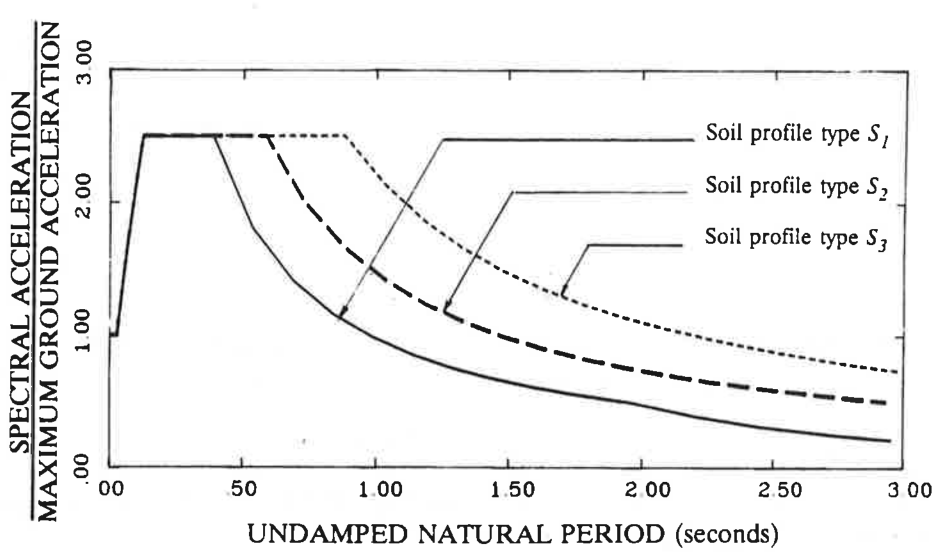


FIGURE C1.4.2-3 Normalized response spectra, damping = 0.05.

The uniform layer on elastic rock sketched in Figure C1.4.2-5 is subjected to a vertically propagating shear wave representing the earthquake. The soil layer is assumed to behave linearly and it has a thickness h, total (saturated) unit weight γ_s, shear wave velocity v_s, and internal damping ratio β_s. The rock has total unit weight γ_r, shear wave velocity v_r, and zero damping. Due to the soil-rock interaction effect, the motion at the soil-rock interface C is different (typically less) from that at the rock outcrop B. Only if the rock is rigid ($v_s = \infty$) are the motions at C and B equal. Of interest here is the ratio between the motions on top of the soil (point A) and on the rock outcrop (point B).

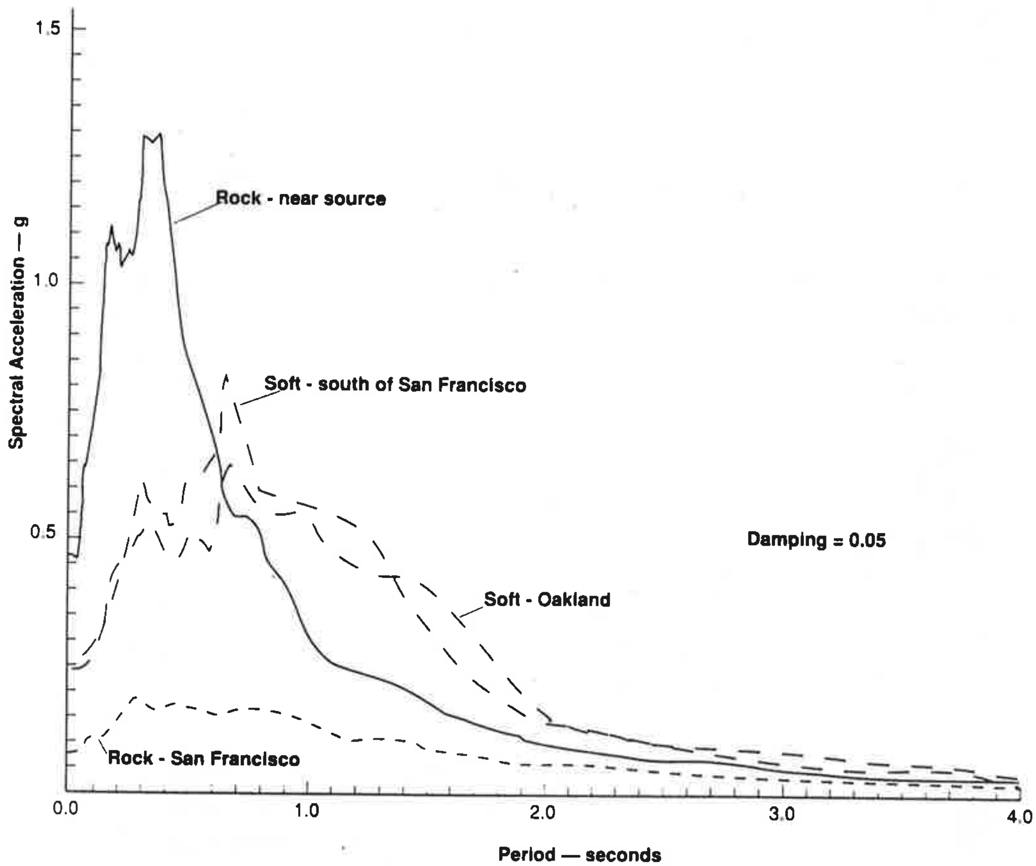


FIGURE C1.4.2-4 Average spectra recorded during 1989 Loma Prieta earthquake at rock sites and soft soil sites (Housner, 1990).

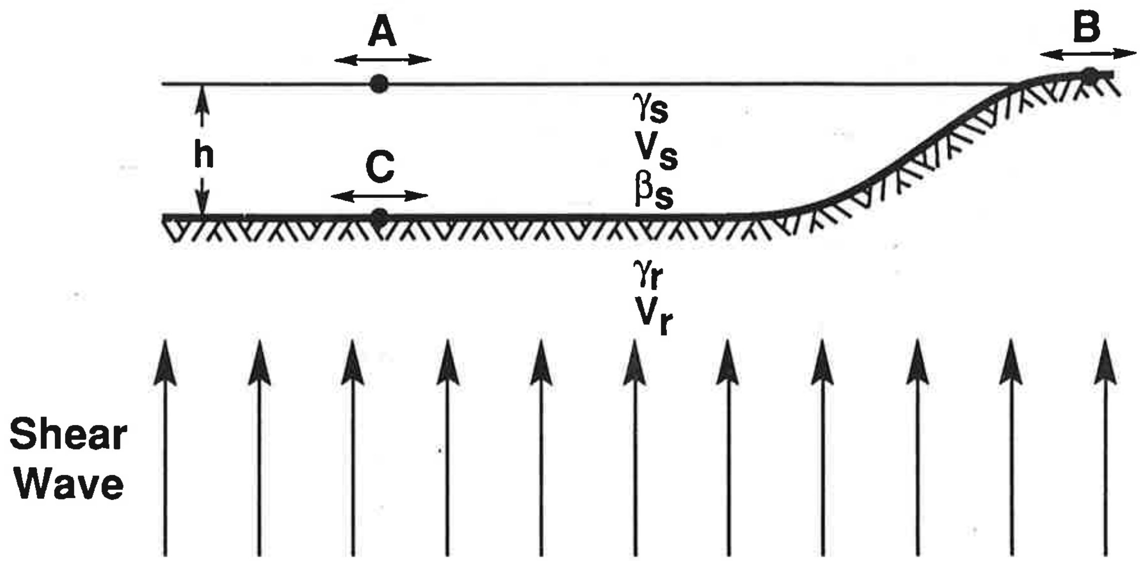


FIGURE C1.4.2-5 Uniform soil layer on elastic rock subjected to vertical shear waves.

When the acceleration at B is a harmonic motion of frequency f (cps) and amplitude a_B, the acceleration at A is also harmonic of the same frequency and amplitude a_A. The amplification ratio a_A/a_B is a function of the ratio of frequencies $f/(v_s/4h)$, of the soil damping β_s, and of the rock/soil impedance ratio which is equal to $\gamma_r v_r / \gamma_s v_s$. Figure C1.4.2-6 presents a_A/a_B calculated for a layer with h = 100 ft (30.5 m), $v_s/4h$ = 1.88 cps, and IR = 6.7 (Roesset, 1977). The maximum amplification occurs essentially at the natural frequency of the layer, $f_{soil} = V_s/4h$, and is approximately equal to:

$$\left(\frac{a_A}{a_B}\right)_{max} \approx \frac{1}{\left(\frac{1}{IR}\right) + \left(\frac{\pi}{2}\right)} \beta_s \qquad \text{(C1.4.2-1)}$$

That is, the maximum soil/rock amplification for steady-state harmonic motion in this simple model depends on two factors--β_s and IR. When $IR = \infty$ (rigid rock), the only way the system can dissipate energy is in the soil and $(a_A/a_B)_{max} = 2/\pi\beta_s$ can be very large. For example, if $IR = \infty$ and $\beta_s = 0.04$, $(a_A/a_B)_{max} = 16$. If IR decreases, the amplification $(a_A/a_B)_{max}$ also decreases. For example, if IR = 15 and β_s = 0.04, the amplification is cut in half, $(a_A/a_B)_{max} = 8$.

Another way of expressing the contribution of the impedance ratio IR in Eq. C1.4.2-1 is as an "additional equivalent soil damping" with a total damping β_{tot} in the system at its natural frequency:

$$\beta_{tot} \approx \beta_s + \left(\frac{2}{\pi IR}\right) \qquad \text{(C1.4.2-2)}$$

Eq. C1.4.2-2 is very important since the maximum amplification $(a_A/a_B)_{max}$ is always inversely proportional to β_{tot}, not only for the case of the uniform layer but also for other soil profiles on rock. β_{tot} always includes an internal damping contribution (β_s) and a second term reflecting the rock-soil impedance contrast IR although the specific definition of IR and the numerical factor $2/\pi$ generally will change depending on the profile. When a soft layer lies on top of a significant thickness of stiffer soil followed by rock, Eq. C1.4.2-2 is still qualitatively valid, but the calculations are more complicated. In that case, the impedance contrast must consider the whole soil profile and, thus, both soft and stiff soils play a role in determining β_{tot} and $(a_A/a_B)_{max}$. Also, the maximum amplification may occur at the natural frequency of the soft layer, of the whole profile, or at some other frequency.

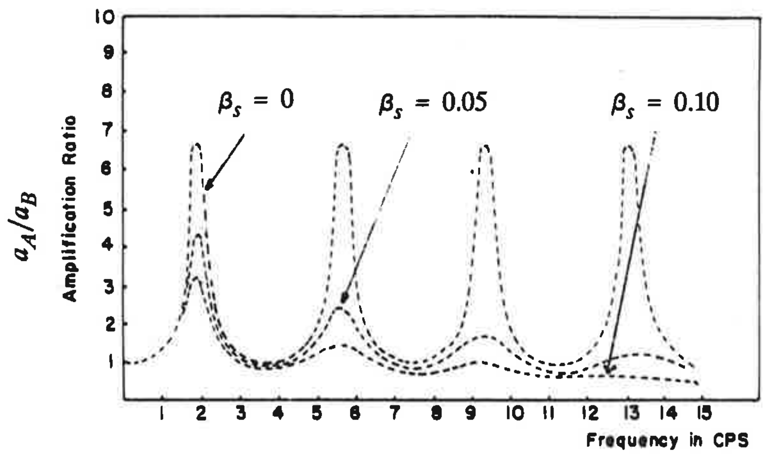


FIGURE C1.4.2-6 Amplification ratio soil/rock for h - 100 ft (30.5 m), V_s = 1.88 cps, and IR = 6.7 (Roesset, 1977).

Two-Factor Approach and the 1992 Site Response Workshop: The recommendations developed during the NCEER/SEAOC/BSSC Site Response Workshop mentioned above were summarized by Rinne and Dobry (1992) and are reprinted as Appendix D of this commentary to provide the reader with a better understanding of the thinking behind the current provisions. Some additional background information taken mostly from the proceedings of that workshop (Martin, 1994) is included below.

As discussed above, soil sites generally amplify more the rock spectral accelerations at long periods than at short periods and, for a severe level of shaking ($A_a \approx A_v \approx 0.4$), the short-period amplification or deamplification is small; this was the basis for the use in the previous versions of the *Provisions* of normalized spectra such as shown in Figure C1.4.2-3 in previous versions of the *Provisions*. However, the evidence that short-period accelerations including the peak acceleration can be amplified several times, especially at soft sites subjected to low levels of shaking, suggested the replacement of the normalized spectrum approach by the two-factor approach sketched in Figure C1.4.2-7. In this approach, adopted in the 1994 *Provisions*, the short-period plateau, of height proportional to A_a, is multiplied by a short-period site coefficient F_a and the curve proportional to A_v/T is multiplied by a long-period site coefficient F_v. Both F_a

and F_v depend on the site conditions and on the level of shaking, defined respectively by the values of A_a and A_v. In both the 1994 and previous versions of the *Provisions*, the descending branch proportional to A_v/T in Figure C1.4.2-7 is replaced by a curve proportional to $A_v/T^{2/3}$, which decreases more slowly with increasing T. This change is based on structural considerations discussed in Sec. 2.3.2 of this *Commentary*.

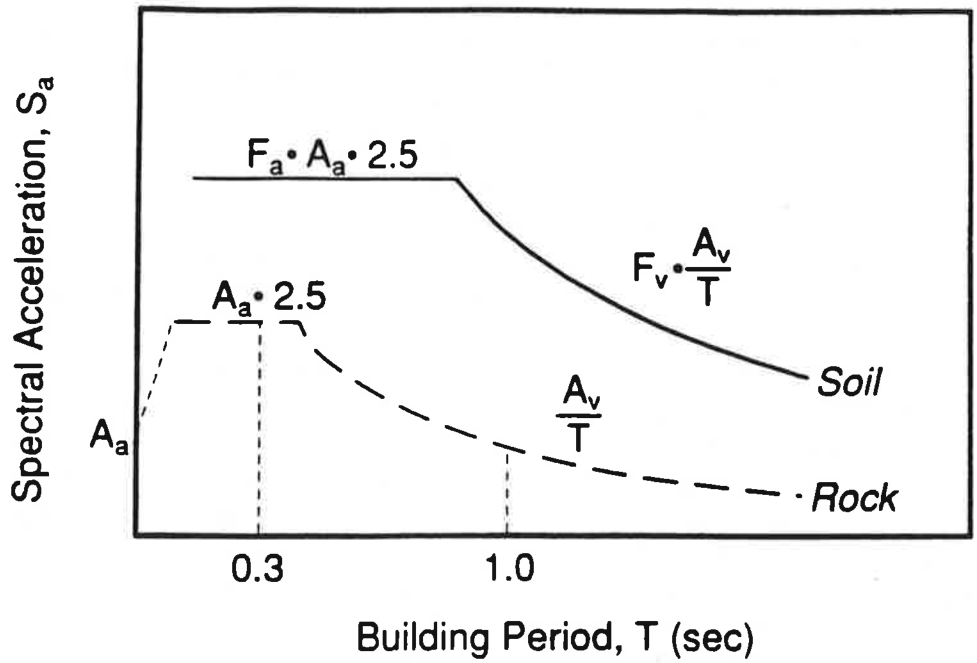


FIGURE C1.4.2-7 Two-factor approach to local site response.

Strong-motion recordings, as obtained from the Loma Prieta earthquake of October 17, 1989, provide important quantitative measures of the *in situ* response of a variety of geologic deposits to damaging levels of shaking. Average amplification factors derived from these data with respect to "firm to hard rock" for short-period (0.1-0.5 sec), intermediate-period (0.5-1.5 sec), mid-period (0.4-2.0 sec), and long-period (1.5-5.0 sec) bands show that a short- and mid-period factor are sufficient to characterize the response of the local site conditions (Borcherdt, 1994). This important result is consistent with the two-factor approach summarized in Figure C1.4.2-7. Empirical regression curves fit to these amplification data as a function of mean shear wave velocity at the site are shown in Figure C1.4.2-8. These curves provide empirical estimates of the site coefficients F_a and F_v as a function of mean shear wave velocity for input ground motion levels near 0.1g (Borcherdt and Glassmoyer, 1993). The empirical amplification factors predicted by these curves are in good agreement with those derived independently based on numerical modelling of the Loma Prieta strong-motion data (Seed et al., 1992) and those derived from parametric studies of several hundred soil profiles (Dobry et al., 1994b). These empirical relations are consistent with theory in that they imply that the average amplification at a site increases as the rock/soil impedance ratio (IR) increases, similar to the trend described by Eq. C1.4.2-1. They also are consistent with observed correlations between amplification and shear velocity for soft clays in Mexico City (Ordaz and Arciniegas, 1992). These short- and mid-period amplification factors implied by the Loma Prieta strong-motion data and related calculations for the same earthquake by Joyner et al. (1994) as well as modelling results at the 0.1g level provided the basis for the consensus values provided in Tables 1.4.2.3a and 1.4.2.3b. Values at higher levels were initially determined from modelling results for soft clays derived by Seed (1994) with values for intermediate soil conditions derived by linear extrapolation. A rigorous framework for extrapolation of the Loma Prieta results consistent with the results in Tables C1.4.2a and C1.4.2b is given in the following paragraph.

Extrapolation of amplification estimates at the 0.1g level as derived from the Loma Prieta earthquake must necessarily be based on laboratory and theoretical modelling considerations because few or no strong-motion recordings have been obtained at higher levels of motion, especially on soft soil deposits. Resulting estimates should be consistent with other relations between large rock and soil motions and local site conditions as summarized in Figure C1.4.2-1. The form of the regression curve in Figure C1.4.2-8 suggests a simple and well defined procedure for extrapolation. It shows that the functional relationship between the logarithms of amplification and mean shear velocity is a straight line (Borcherdt, 1993). Consequently, as the amplification factor for "firm to hard" rock is necessarily unity, the extrapolation problem is determined by specification of the amplification factors at successively higher levels of motion for the soft-soil site class. For input ground motion levels near 0.1g, Borcherdt (1993) began with amplification levels specified by the empirical regression curves (Figure C1.4.2-8) for the Loma Prieta strong-motion data. Higher levels of motion were inferred from laboratory and numerical modelling results (Seed et al., 1992; Dobry et al., 1994a). The resulting short-period (F_a) and mid-period (F_v) site coefficients as a function of mean shear velocity (v--labelled $\overline{v}_s$ elsewhere in this *Commentary* and in the *Provisions*) and input ground motion level (I_a) specified with respect to "firm to hard" rock are given in Figure C1.4.2-9 and plotted with logarithmic scales. These expressions state that the average amplification at a site is equal to the "rock-soil" impedance ratio raised to an exponent (*ma* or *mv*). These exponents are defined as the slope of the straight line determined by the logarithms of the amplification factors and the shear velocities for the soft-soil and the "firm to hard" rock site classes at the specified input ground motion level (Borcherdt, 1993). The equations in Figure C1.4.2-9 provide a framework to illustrate a simple procedure for derivation of amplification factors that are in general agreement with the consensus values included in Tables 1.4.2.3a and 1.4.2.3b of the *Provisions*. However, the numbers in these tables of the *Provisions* are not necessarily identical to the equations' predictions due to other considerations discussed during the consensus process.

Extensive site response studies using both equivalent linear and nonlinear programs were conducted by several groups as listed by Rinne and Dobry (1992). The main objectives of these studies were to generalize the experience of well documented earthquakes such as Loma Prieta and Mexico City to a variety of site conditions and earthquake types and levels of shaking. Some results obtained by Dobry et al. (1994a) are reproduced in Figures C1.4.2-10 to C1.4.2-12.

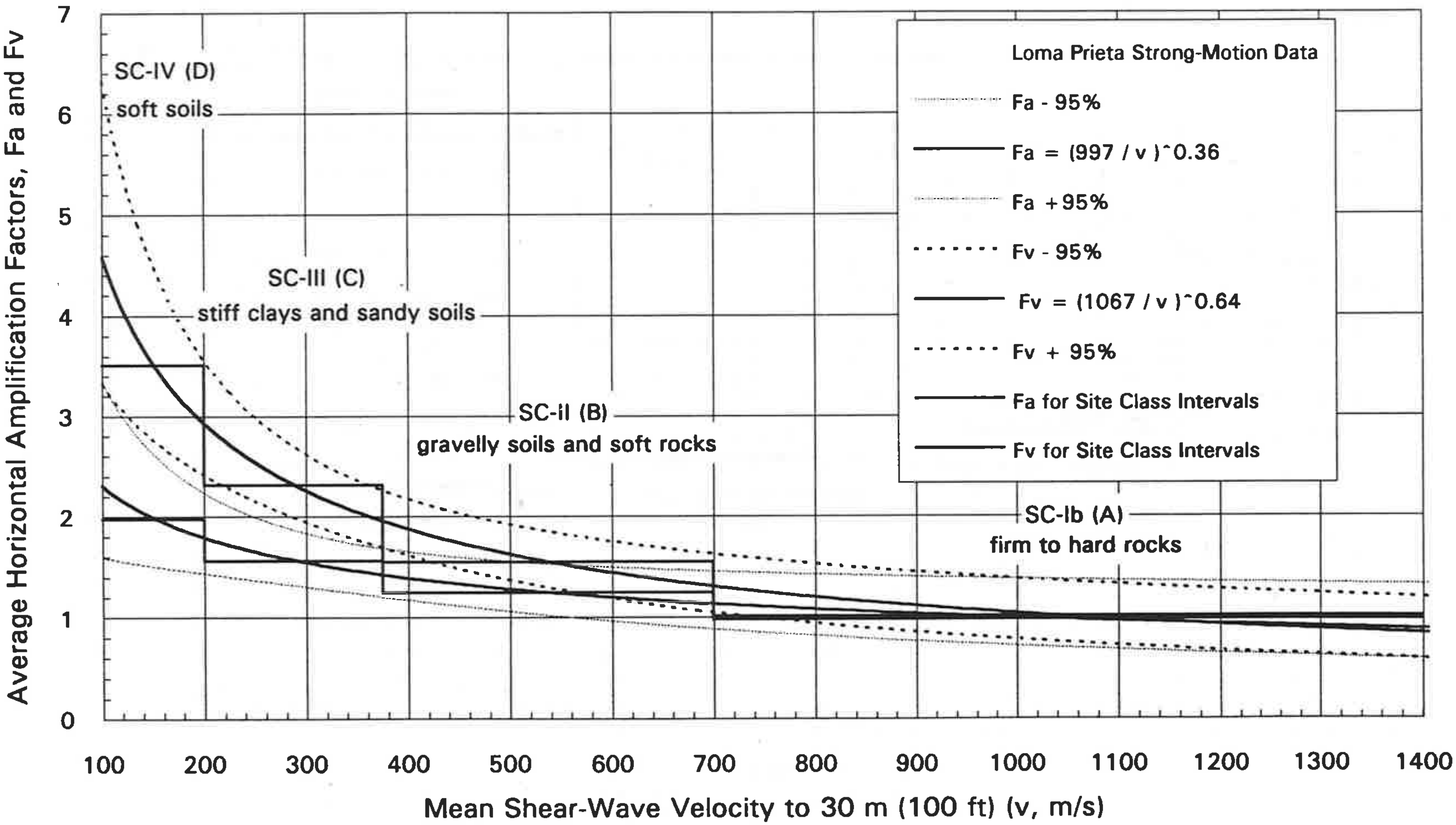
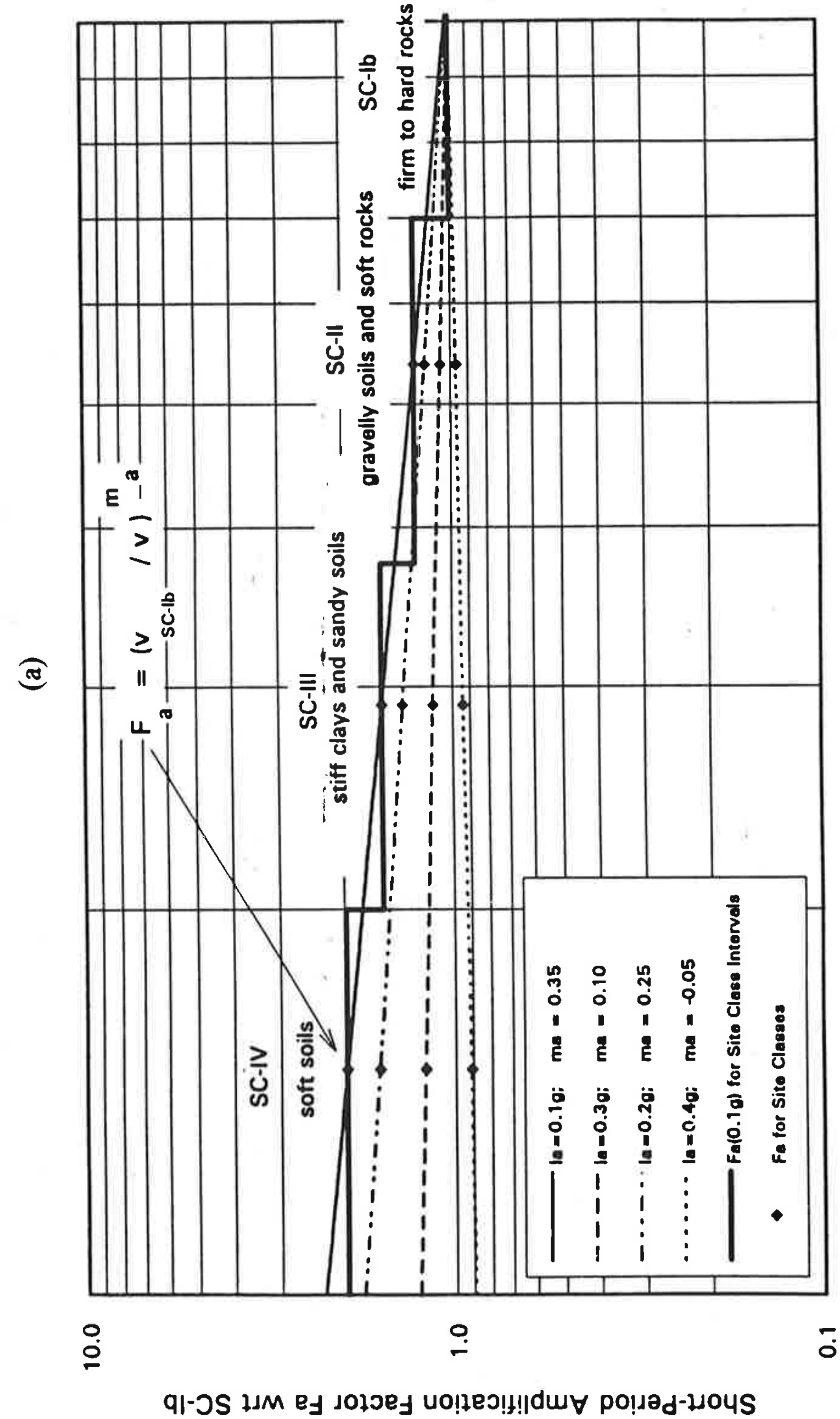


FIGURE C1.4.2-8 Short-period F_a and mid-period F_v amplification factors with respect to "firm to hard" rock plotted as a continuous function of mean shear wave velocity using the regression equations derived from the strong-motion recordings of the Loma Prieta earthquake. The 95 percent confidence intervals for the ordinate to the true population regression line and the amplification factors for the simplified site classes also are shown (Borcherdt, 1994).


(a)
F = (v / v) m
a SC-Ib a
SC-IV
soft soils
SC-III
stiff clays and sandy soils
SC-II
gravelly soils and soft rocks
SC-Ib
firm to hard rocks
Ia=0.1g; ma = 0.35
Ia=0.3g; ma = 0.10
Ia=0.2g; ma = 0.25
Ia=0.4g; ma = -0.05
Fa(0.1g) for Site Class Intervals
Fa for Site Classes
10.0
1.0
0.1
100
1000
Short-Period Amplification Factor Fa wrt SC-Ib

(b)

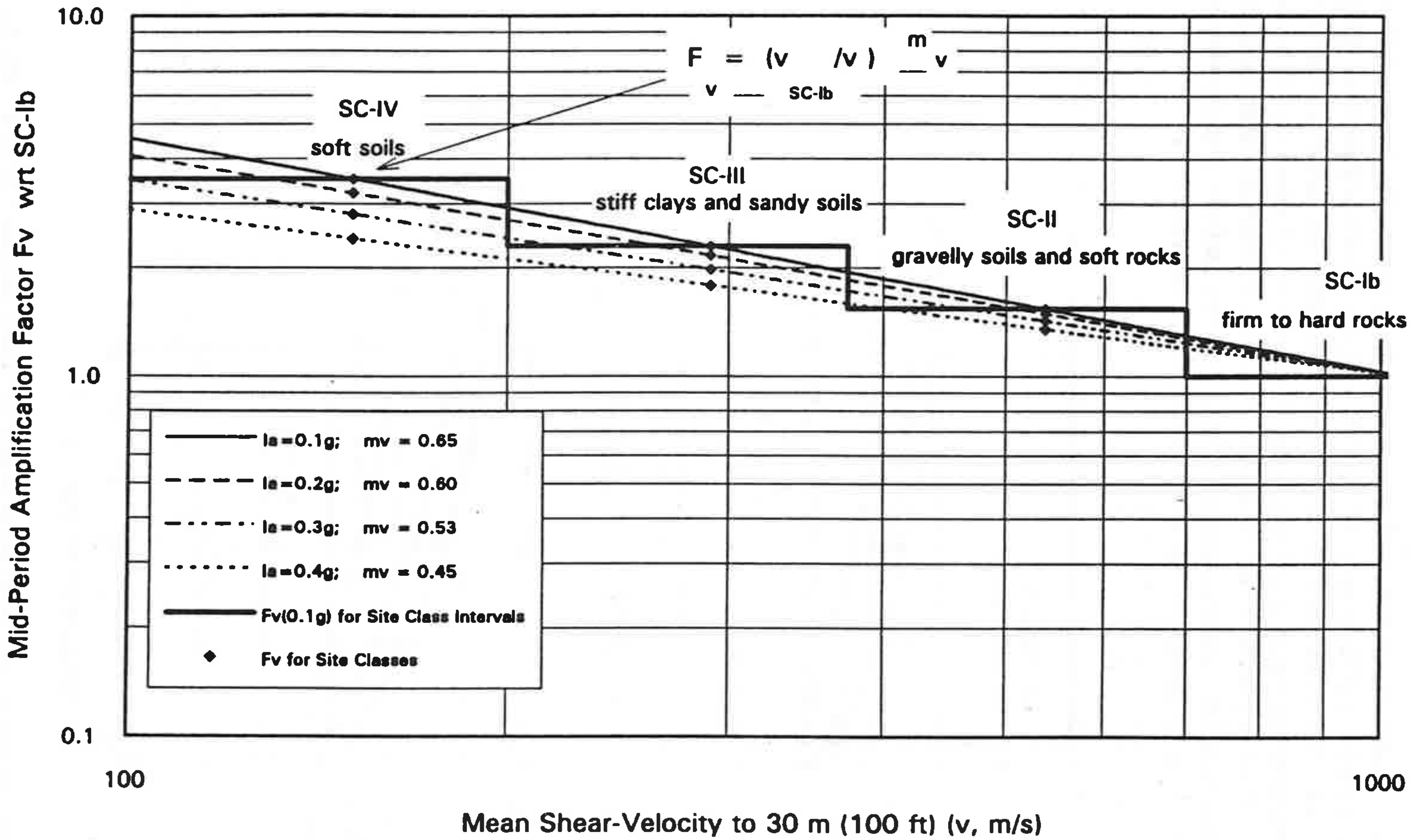


FIGURE C1.4.2-9 (a) short-period F_a and (b) mid-period F_v amplification factors with respect to "firm to hard" rock (SC-Ib) plotted with logarithmic scales as a continuous function of mean shear wave velocity using the indicated equations for specified levels of input ground motion. The equations correspond to straight lines determined by the points defined as the logarithms of the amplification factors and shear velocities for the "soft-soil" and "firm to hard" rock site classes. The amplification factors for the "soft-soil" site class are based on strong motion recordings at the 0.1g level and on numerical modelling and expert opinion results for higher levels of motion. The exponents *ma* and *mv* are given by the slope of the indicated straight lines. Amplification factors with respect to SC-Ib for the simplified site classes are shown for the corresponding mean shear wave velocity interval for input ground motion levels near 0.1g (Borcherdt, 1993).

Figure C1.4.2-10 presents values of peak amplification at long periods for soft sites (labelled RRS_{max} in the figure) calculated using the equivalent linear approach as a function of the plasticity index (*PI*) of the soil, rock wave velocity v_r, and for weak and strong shaking. The effect of PI is due to the fact that soils with higher PI exhibit less stress-strain nonlinearity and a lower damping β_s (Vucetic and Dobry, 1991). For $A_a = A_v = 0.1$g, $v_r = 4{,}000$ ft/sec (1220 m/s) and PI = 50, roughly representative of Bay area soft sites in the Loma Prieta earthquake, RRS_{max} = 4.4, which coincides with the upper part of the range backfigured by Borcherdt from the records. Note the reduction of this value of RRS_{max} from 4.4 to about 3.3 when $A_a = A_v = 0.4$g due to soil nonlinearity. Evidence such as this is used in the 1994 *Provisions* to extrapolate values of F_a and F_v at low levels of shaking--based on both analysis and observations--to high levels of shaking for which no observations on soft sites currently are available.

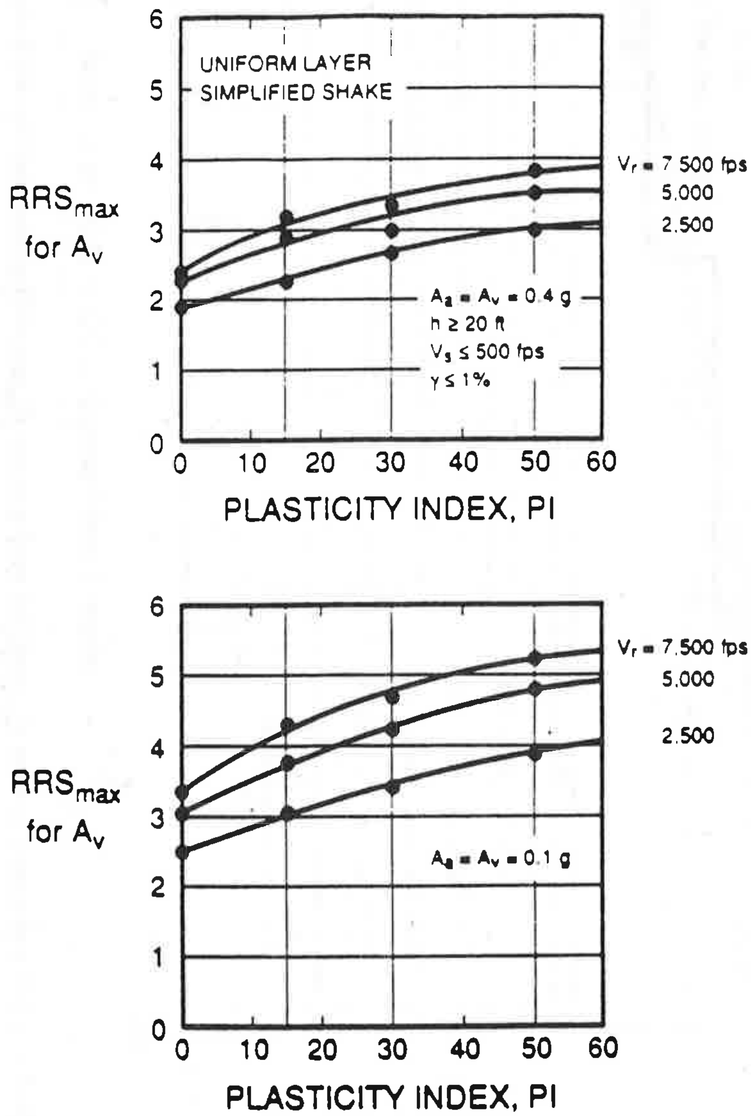


FIGURE C1.4.2-10 Summary of uniform layer analyses using simple SHAKE (Dobry et al., 1994a)

Specific equivalent linear runs using the SHAKE program corresponding to the same situation are included in Figure C1.4.2-11 while Figure C1.4.2-12 summarizes and compares them with calculations by Joyner et al. (1994) from the Loma Prieta records on soft sites similar to the work by Borcherdt mentioned above.

Another important observation from analytical results such as shown in Figure C1.4.2-11 is that the values of RRS_{max} are about 20 percent higher for soft sites on "hard rock"--characterized by v_r = 7,500 ft/sec (2290 m/s)--than for soft sites on "regular rock" corresponding to v_r = 4,000 ft/sec (1220 m/s). This is again the impedance ratio effect previously discussed. Separate studies indicate that earthquake motions on outcrops of "hard rock" tend to be smaller than on outcrops of "regular rock" by 10 to 40 percent at both short and long periods (except at very small periods under about 0.2 sec where the reverse may be true); see Su et al. (1992) and Silva (1992). On the basis of these studies and observations, the 1994 *Provisions* incorporate the difference between "regular" rock (B) and "hard" rock of $\overline{v}_s$ > 5,000 ft/sec (1520 m/s) by defining a new "hard rock" site category (A) and assigning to it site factors $F_a = F_v = 0.8$.

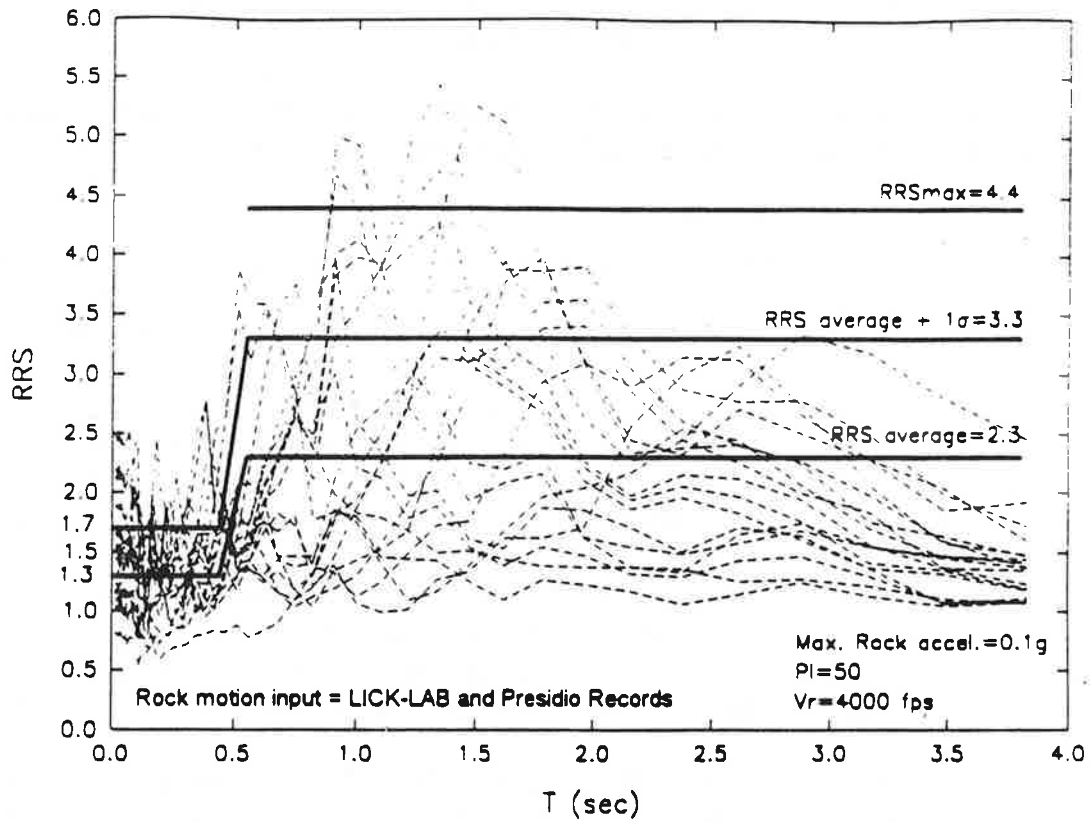


FIGURE C1.4.2-11 Summary of uniform layer analyses using SHAKE program, $h \geq 50$ ft (15.2 m) (Dobry et al., 1994a).

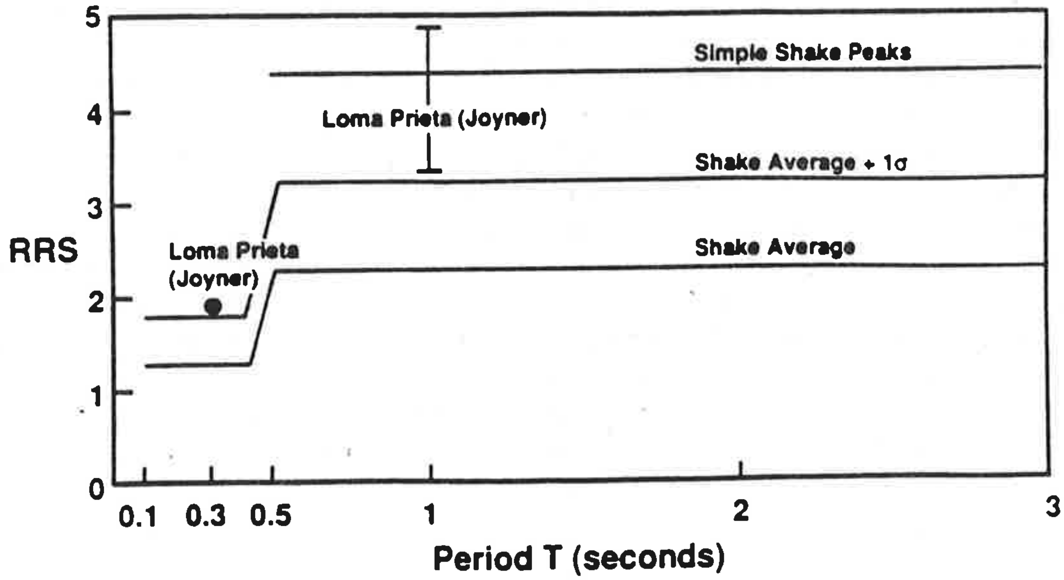


FIGURE C1.4.2-12 Comparison between RRS SHAKE program results and those obtained by Joyner et al. (1994) for the 1989 Loma Prieta event (Dobry et al., 1994a).

Use of Geotechnical Parameters Instead of $\bar{v}_s$: Based on the studies and observations discussed above, the site categories in the 1994 *Provisions* are defined in terms of the average shear wave velocity in the top 100 ft (30.5 m) of the profile, $\bar{v}_s$. If the shear wave velocities are available for the site, they should be used.

However, in recognition of the fact that in many cases the shear wave velocities are not available, alternative definitions of the site categories also are included in the 1994 *Provisions*. They use the standard penetration resistance for cohesionless soil layers and the undrained shear strength for cohesive soil layers. These alternative definitions are rather conservative since the correlation between site amplification and these geotechnical parameters is more uncertain than that with $\bar{v}_s$. That is, there will be cases when the values of F_a and F_v will be smaller if the site category is based on $\bar{v}_s$ rather than on the geotechnical parameters. Also, the reader must not interpret the site category definitions as implying any specific numerical correlation between shear wave velocity on the one hand and standard penetration or shear strength on the other.

1.4.3 SEISMIC HAZARD EXPOSURE GROUPS: Historically, building code occupancy classifications are based primarily on fire-safety considerations. It was concluded, however, that these traditional classifications would at least in part reflect some considerations contrary to good seismic design. Thus, it was decided that 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 involve evaluation of parameters consisting of, but not limited to the number, age, and condition of the persons normally expected to be within or without the immediate environs of the building; the size, height, and area of the building; 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; and the varying degree of built-in or brought-in hazards based on possible use of the building. Accordingly, early in the development of the preliminary version of the *Provisions* occupancy types were regrouped and expanded to cover the 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*).

In terms of post-earthquake recovery and redevelopment, certain types of occupancies are vital to public needs, and these special occupancies were identified and given specific recognition (i.e., 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 also was given to the preservation of strategic contents in distinct building types (e.g., storage facilities for medical supplies, critical foodstuffs, and other emergency materials). 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, however, 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 and, therefore, no separate category of warehousing was required for the storage of critical materials. Thus, nine occupancy groups, A through I, were identified 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 identified 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. The intermediate grouping involved the following: Group I--fire, police, hospitals; Group II--public assembly, open air stands, day care, schools, colleges, retail stores, shopping centers, offices, hotels, apartments, emergency vehicles, power utilities; Group III--restrained occupants, nurseries (nonambulatory); Group IV--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; and Group V--private garages, sheds, barns.

The occupancy grouping used in the 1985 Edition of the *Provisions* resulted from a logical consolidation of the grouping, 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 and also appear in the 1991 and 1994 Editions. It is felt that this grouping can be augmented as local conditions warrant.

Specific consideration is given to Group III, essential facilities required for post-earthquake recovery. Also included are buildings housing substances deemed to be dangerous to the public if they are released. Added in the 1991 Edition was a flag to urge consideration of the need for utility services after an earthquake.

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. Note that the safety afforded the occupants in an assembly or living (sleeping) environment should be greater than that provided for general uses. The potential density of public assembly uses in terms of number of people warrant an extra level of care. The same is true of multifamily residential buildings where people spend a majority of their time. Because of the nature of the occupants of secondary schools through day-care centers, the level at which protection is warranted is less than those where individuals are relatively self-sufficient in responding to an emergency. The sleeping environment of the multifamily residential uses is considered equivalent to the potential mobility deficiencies within secondary schools through day-care centers.

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.

In buildings with multiple uses, the 1988 Edition of the *Provisions* required that the building be assigned the classification of the highest group occupying 15 percent or more of the total building area. This was changed in the 1991 Edition to require the building to be assigned to the highest group present.

Such assignments also must 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 change of use in a building of Seismic Hazard Exposure Group I involves the introduction of a Group II occupancy, the *Provisions* requires that the building conform to the requirements for Group II unless the building is in a seismic map area having an effective peak velocity-related acceleration (A_v) value of less than 0.15.

Consideration has been given to reducing the number of groupings by combining Groups I and II and leaving Group III the same as is stated above; however, the consensus of those involved in the *Provisions* development and update efforts to date is that such a merging would not be responsive to the relative life hazard problems.

1.4.4 SEISMIC PERFORMANCE CATEGORY: 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 effective peak velocity-related 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.

1.4.5 SITE LIMITATION FOR SEISMIC PERFORMANCE CATEGORY E: 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 avoided.

1.5 ALTERNATIVE 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. If accepted standards are lacking, it is strongly recommended that 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 quality assurance been exercised. The remarkable performance during earthquakes by California schools constructed since 1933 is due in great part to the rigorous supervision of design and construction by the Office of the State Architect as required by state law. The *Provisions* is written to rely heavily on the concept of special quality controls to ensure good construction.

For buildings located in areas of seismic risk and subject to potential earthquake ground motion, good quality control and verification are especially important because of the serious consequences of failure and the unique, generally more complex, nature of building design and construction when required to resist earthquake forces. The weakest links in the seismic force resisting systems are affected by lateral forces. Building failures generally can be traced directly to a lack of quality control during design or construction or both when these links or details are slighted.

The building designer specifies the quality assurance requirements, the prime contractor exercises the control to achieve the desired quality, and the owner monitors the construction process through special inspection and testing 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 or her own employees.

These provisions are concerned with those components that affect building performance during an earthquake or that may be adversely affected by earthquake motions as specified of other sections of the *Provisions*. The requirements under Sec. 1.5 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 (QAP) 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 or she 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 have been included in the model codes for years 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 is a "person approved by the regulatory agency as being qualified to perform special inspection for the category of work involved." 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, using certified inspectors or technicians qualified by recognized industry organizations as approved by the regulatory 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 assurance 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 in Seismic Performance Categories D or E.

1.6.2.9 Mechanical and Electrical Components: It is anticipated that the minimum requirements for mechanical and electrical 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 for selected power, piping, and ductwork components in all Seismic Performance Categories except A, and for all other electrical equipment in Seismic Performance Category E.

1.6.3 TESTING: The specified testing of the structural materials follows procedures and tests long established by industry standards. The acceptance criteria should be included in the project construction documents.

1.6.3.5 Mechanical and Electrical Equipment The facility designer should consider requirements to demonstrate the seismic performance of mechanical or electrical components critical to the post-earthquake life safety of the occupants. Any requirements should be clearly indicated on the contract drawings and/or specifications. Any currently accepted technology should be acceptable to demonstrate compliance with the requirements.

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 the inspector's 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, having 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 and the special testing conducted by the testing agency.

REFERENCES

American Society of Civil Engineers Working Group on Quantification of Uncertainties of the Committee on Dynamic Analysis of the Committee on Nuclear Structures and Materials of the Structural Division. 1986. *Uncertainty and Conservatism in the Seismic Analysis and Design of Nuclear Facilities.* New York: ASCE.

Ang, A. H., and N. M. Newmark. 1977. *A Probabilistic Seismic Safety Assessment of the Diablo Canyon Nuclear Power Plant*, report to the U.S. Nuclear Regulatory Commission.

Applied Technology Council. 1974. *An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings*, Report ATC-2. San Francisco, California: Applied Technology Council.

Applied Technology Council. 1978. *Tentative Provisions for the Development of Seismic Regulations for Buildings*, Report ATC 3-06. San Francisco, California: Applied Technology Council.

Blume, J. A., R. L. Sharpe, and J. Dalal. 1973. *Recommendations for Shape of Earthquake Response Spectra*, Report prepared for the Directorate of Licensing. Washington, D.C.: U.S. Atomic Energy Commission.

Borcherdt, R. D. 1994. "Simplified Site Classes and Empirical Amplification Factors for Site-Dependent Code Provisions." In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Borcherdt, R. D. 1993. "General Methodology for Estimating Free-Field Site Specific Response Spectra." In Minutes of the Building Seismic Safety Council meeting, April 5-6, 1993, San Francisco, California.

Borcherdt, R. D., and G. Glassmoyer. 1993. "Influence of Local Geology and Weak Ground Motions in the San Francisco Bay Region, California, and Their Implications for Site-Specific Code Provisions." In *The Loma Prieta Earthquake of October 17, 1989 - Strong Ground Motion*, USGS Professional Paper 1551-A, edited by R. D. Borcherdt.

Dobry, R., G. M. Martin, E. Parra, and A. Bhattacharyya. 1994a. "Development of Site-Dependent Ratios of Elastic Response Spectra (RRS) and Site Categories for Building Seismic Codes." In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Dobry, R., G. M. Martin, E. Parra, and A. Bhattacharyya. 1994b. *Study of Ratios of Response Spectra Soil/Rock and of Site Categories for Seismic Codes.* Buffalo, New York: National Center for Earthquake Engineering Research.

Electric Power Research Institute. 1988. *Seismic Hazard Methodology for the Central and Eastern United States*, Technical Report NP-4726-A, 1988 Edition.

Hayashi, S., Tsuchida, and E. Kurata. 1971. "Average Response Spectra for Various Subsoil Conditions." Paper presented at the Third Joint Meeting of the U.S.-Japan Panel on Wind and Seismic Effects, UNJR, Tokyo, May 10-12.

Housner, G. W. 1990. *Competing Against Time: Report to Governor George Deukmejian from the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake*.

Idriss, I. M. 1990a. "Response of Soft Soil Sites During Earthquakes." In *Proceedings of the Symposium to Honor Professor H. B. Seed*, Berkeley.

Idriss, I. M. 1990b. "Influence of Local Site Conditions on Earthquake Ground Motions." In *Proceedings of the 4th U.S. National Conference on Earthquake Engineering*, Palm Springs, Vol. 1, pp. 55-57.

Jacob, K. 1990. "Seismic Hazards and the Effects of Soils on Ground Motions for the Greater New York City Metropolitan Region." In *Geotechnical Aspects of Seismic Design in the N.Y.C. Metropolitan Areas; Risk Assessment, Code Requirements and Design Techniques*, ASCE Metropolitan Section, page 24, New York, New York, November 13-14.

Jacob, K. 1991. "Seismic Zonation and Site Response: Are Building-Code Soil Factors Adequate to Account for Variability of Soil Conditions Across the US?" In *Proceedings of the 4th International Conference on Seismic Zonation*, Vol. II, pp. 695-702. Oakland, California: Earthquake Engineering Research Institute.

Joyner, W. B., T. E. Fumal, and G. Glassmoyer. 1994. "Empirical Spectral Response Ratios for Strong Motion Data from the 1989 Loma Prieta, California, Earthquake." In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Kennedy, R. P., S. A. Short, J. R. McDonald, M. W. McCann, R. C. Murray, and J. R. Hill. 1990. *Design and Evaluation Guidelines for Department of Energy Facilities Subjected to Natural Phenomena Hazards*, prepared for the Office of the Assistant Secretary for Environment, Safety, and Health, Office of Safety Appraisals, U.S. Department of Energy.

Mark, R. K. 1977. "Application of Linear Statistical Models of Earthquake Magnitude Versus Fault Length in Estimating Maximum Expectable Earthquakes." *Geology* 5:464-466.

Martin, G. M., Editor. 1994. *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20.

Milne, W. G., and A. G. Davenport. 1969. "Distribution of Earthquake Risk in Canada." *Bulletin of the Seismological Society of America* 59(2):754-779.

Mohraz, B. 1976. "A Study of Earthquake Response Spectra for Different Geological Conditions." *Bulletin of the Seismological Society of America* 66 (3):915-935.

National Bureau of Standards and Building Seismic Safety Council. 1982. *Amendments to ATC 3-06, Tentative Provisions for the Development of Seismic Regulations for Buildings, for Use in Trial Designs*. Washington, D.C.: U.S. Department of Commerce.

Newmark, N. M. 1973. *A Study of Vertical and Horizontal Spectra*, Report WASH-1255. Washington, D.C.: U.S. Atomic Energy Commission, Directorate of Licensing.

Ordaz, M., and Arciniegas. 1992. Personal communication to Ricardo Dobry.

Rinne, E., and R. Dobry. 1992. Preliminary Site Recommendations, Memorandum to Roland Sharpe, Chairman TS 2, Building Seismic Safety Council, December 11.

Roesset, J. M. 1977. "Soil Amplification of Earthquakes." In *Numerical Methods in Geotechnical Engineering*, edited by C. S. Desai and J. T. Christian, Chapter 19, pp. 639-682. New York: McGraw-Hill.

Schnabel, P., and H. B. Seed. 1973. "Acceleration in Rocks for Earthquakes in the Western United States." *Bulletin of the Seismological Society of America* 63:501-516.

Schnabel, P. B., J. Lysmer, and H. B. Seed. 1972. *SHAKE, a Computer Program for Earthquake Analysis of Horizontally Layered Soils*, Report EERC 72-12. Berkeley, California: Earthquake Engineering Research Center.

Seed, R. B. 1994. In *Proceedings of the NCEER/SEAOC/BSSC Workshop on Site Response During Earthquakes and Seismic Code Provisions*, University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.

Seed, R. B., S. E. Dickenson, G. A. Rau, R. K. White, and C. M. Mok. 1992. "Observations Regarding Seismic Response Analyses for Soft and Deep Clay Sites."

Seed, R., S. E. Dickenson, M. F. Riemer, J. D. Bray, N. Sitar, J. K. Mitchell, I. M. Idriss, R. E. Kayen, A. Kropp, L. F. Harder Jr., and M. S. Power. 1990. *Preliminary Report on the Principal Geotechnical Aspects of the October 17, 1989, Loma Prieta Earthquake*. Report UCB/EERC-90/05. Berkeley, California: EERC.

Seed, H. B., R. Murarka, J. Lysmer, and I. M. Idriss. 1976a. "Relationships Between Maximum Acceleration, Maximum Velocity, Distance from Source and Local Site Conditions for Moderately Strong Earthquakes." *Bulletin of the Seismological Society of America* 66 (4):1323--1342.

Seed, H. B., M. P. Romo, J. I. Sun, A. Jaime, and J. Lysmer. 1988. "The Mexico Earthquake of September 19, 1985--Relationships Between Soil Conditions and Earthquake Ground Motions." *Earthquake Spectra* 4(4):687-729.

Seed, H. B., C. Ugas, and J. Lysmer. 1976b. "Site Dependent Spectra for Earthquake-Resistant Design." *Bulletin of the Seismological Society of America* 66 (1):221-244.

Seed, H. B., C. Ugas, and J. Lysmer. 1974. *Site-Dependent Spectra for Earthquake-Resistant Design*, Report EERC 74-12. Berkeley, California: EERC.

Silva, W. 1992. Personal communication to Ricardo Dobry.

Structural Engineers Association of California. 1970. *Report of the Ad Hoc Committee on Cost of Design for Earthquakes.* Washington, D.C.: Office of Science and Technology, Task Force on Earthquake Hazard Reduction.

Su, F., K. Aki, T. Teng, Y. Zeng, S. Koyanagi, and K. Mayeda. 1992. "The Relation Between Site Amplification Factor and Surficial Geology in Central California." *Bulletin of the Seismological Society of America* 82(2):580-602.

Thenhaus, P. C., D. M. Perkins, S. T. Algermissen, and S. L. Hanson. 1987. "Earthquake Hazard in the Eastern United States: Consequences of Alternative Seismic Source Zones." *Earthquake Spectra* 3(May).

Thenhaus, P. C., J. I. Ziony, W. H. Diment, M. G. Hopper, D. M. Perkins, S. L. Hanson, and S. T. Algermissen. 1985. "Probabilistic Estimates of Maximum Seismic Horizontal Ground Acceleration on Rock in Alaska and the Adjacent Continental Shelf." *Earthquake Spectra* 1(2):286-306.

U.S. Nuclear Regulatory Commission. 1976. *Reactor Safety Study,* Report WASH-1400. Washington, D.C.: U.S. Nuclear Regulatory Commission.

Vucetic, M., and R. Dobry. 1991. "Effect of Soil Plasticity on Cyclic Response." *Journal of Geotechnical Engineering*, ASCE, 117(1):89-107.

Whitman, R., Editor. 1992. *Proceedings of the Site Effects Workshop, October 24-24, 1991*, Report NCEER-92-0006. Buffalo, New York: National Center for Earthquake Engineering Research.

Chapter 2 Commentary

STRUCTURAL DESIGN CRITERIA, ANALYSIS, AND PROCEDURES

2.1 REFERENCE DOCUMENT: ASCE 7 is referenced for the combination of earthquake loadings with other loads as well as for the computation of other loads; it is not referenced for the computation of earthquake loads.*

2.2 STRUCTURAL DESIGN REQUIREMENTS:

2.2.1 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 the prescribed forces, will 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. 2.3.7.) Sec. 2.2.7 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 earlier 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 6. For other structural materials that do not have their sectional yielding capacities as easily defined, modifiers to working stress values are provided in Chapters 8 and 9.

These provisions contemplate a seismic force 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 C2.2.1, 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 C2.2.1 is the load-deflection curve including the P-delta

* It should be noted that the 1993 Edition of ASCE 7 is essentially the same as the 1991 Edition of the *Provisions* and that the 1995 Edition of ASCE 7 is expected to reflect the 1994 Edition of the *Provisions*.

effects. The dash-dot line is the elastoplastic curve that results with certain systems and materials.

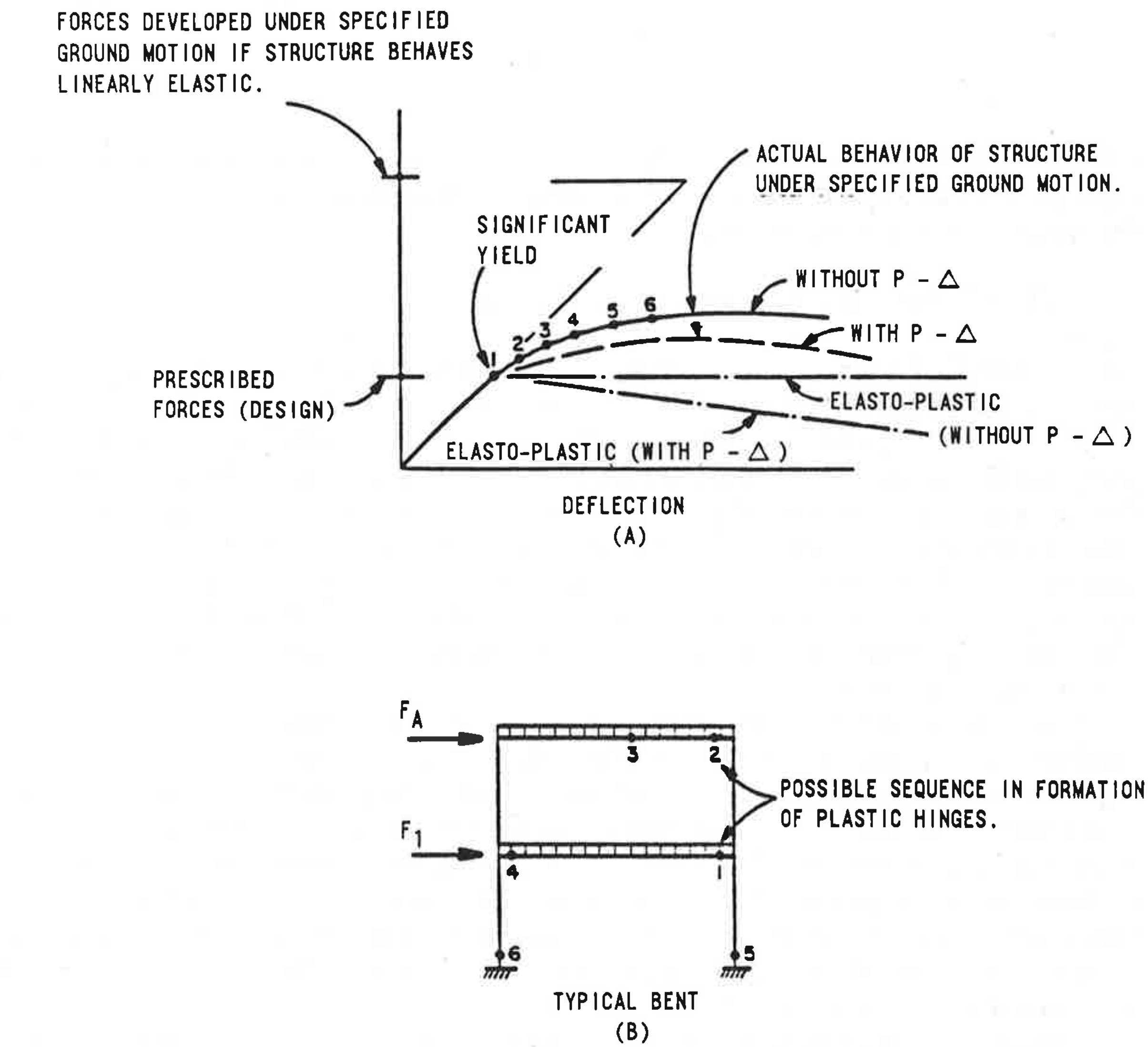


FIGURE C2.1.1 Formation of plastic hinges.

The response modification factor, R, and the C_d value for deflection amplification (Table 2.2.2) as well as the criteria for story drift including 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 had an entirely linearly elastic response 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 various materials and systems in past earthquakes.

Note that the value of R increases with higher toughness and damping whereas the design seismic force decreases. In Eq. 2.3.2.1-1, R is used in the denominator of the term to calculate the seismic base shear.

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 Eq. 2.3.2.1-1 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 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. 2.2.5 for each Seismic Performance Category and the more stringent drift limits in Table 2.2.7.

Sec. 2.2.1 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 it often is overlooked by those inexperienced in earthquake engineering.

2.2.2 STRUCTURAL FRAMING SYSTEMS: For purposes of these seismic analyses and design provisions, building framing systems are grouped in the structural system categories shown in Table 2.2.2. 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 the various types of vertical components in the seismic force resisting system.

In selecting a 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 2.2.2 continue to be reviewed in light of recent research results.

In the selection of the R values for the various systems, consideration has been 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).

A bearing wall system refers to that structural support system wherein major load-carrying columns are omitted and the walls 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 loads. 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 frame walls with shear panels" is intended to cover wood or steel stud wall systems with finishes other than masonry veneers.

A building frame system is a system in which the gravity loads are carried primarily by a frame supported on columns rather than by bearing walls. Some minor portions of the gravity load may 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 frame 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 force 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. Moment resisting frames may be either ordinary, intermediate, or special moment frames as indicated in Table 2.2.2 and limited by the Seismic Performance Categories.

Special moment frames must meet all the design and detail requirements of Sec. 5.6.3, 6.3.3, or 8.1.2. The ductility requirements for these frame systems are required in areas where high seismic hazards are anticipated. Intermediate moment frames of concrete must meet the requirements of Sec. 6.3.2. 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. 2.2.2 (Table 2.2.2) requires moment frames in Categories D or E greater than 160 ft and 100 ft in height, respectively, to be special moment frames.

Provisions for composite steel-concrete systems are new in the 1994 Edition. The R and C_d values for the composite systems in Table 2.2.2 are similar to those for comparable systems of structural steel and reinforced concrete. The values shown in Table 2.2.2 are only allowed when the design and detailing provisions for composite structures in Chapter 7 are followed.

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.

2.2.2.1 Dual System: 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 resisting system that is a moment frame complying with the requirements of Sec. 5.6 and Sec. 6.3.1 or 6.3.2. 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.

2.2.2.2 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 C_d values. The intent of Sec. 2.2.2.2.1 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. 2.2.2.2.2 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 force resisting system.

2.2.2.3 - 2.2.2.5 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. 2.2.5 and Chapters 5 through 9. 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 6 and 8. Limitations regarding the use of different structural systems are given for Categories D and E.

2.2.2.4 Seismic Performance Category D: Sec. 2.2.2.4 covers Category D, which compares roughly to California design practice for normal buildings other than hospitals. According to the requirements of Chapters 5 and 6, 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 ft (49 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 ft (49 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 ft (49 m) and 240 ft (73 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.

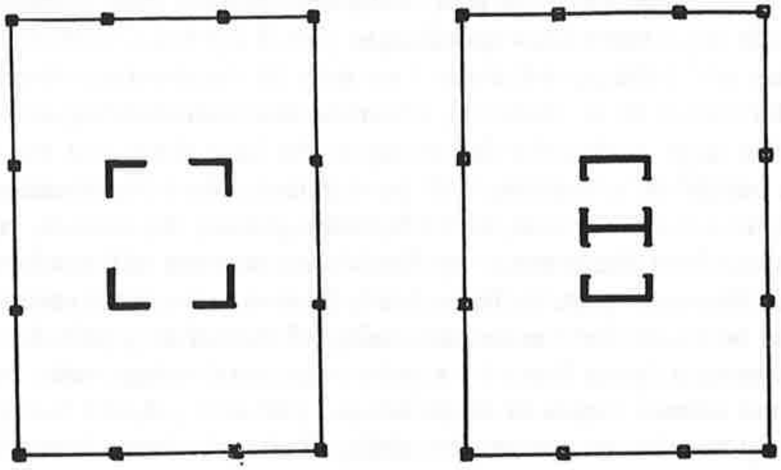
These provisions require that buildings in Category D over 160 ft (49 m) in height have one of the following seismic force resisting systems:

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 the Glossary, 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 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 2.2.2). 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 Sec. 2.3, 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.

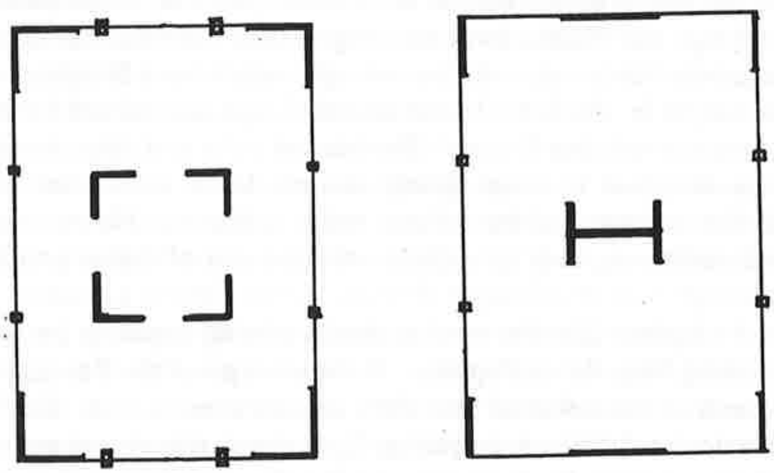
3. The use of a shear wall (or braced frame) system of cast-in-place concrete or structural steel up to a height of 240 ft (73 m) is permitted if, and only if, braced frames or shear walls in any plane do not resist more than 33 percent of the seismic design force including torsional effects. The intent 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 a structural system with lateral force resistance concentrated in the interior core (Figure C2.2.2.4-1) 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 force resistance distributed across the entire building (Figure C2.2.2.4-2). 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.



(a) (b)

FIGURE 2.2.2.4-1 Arrangement of shear walls and braced frames--not recommended. Note that the heavy lines indicate shear walls and/or braced frames.



(a) (b)

FIGURE C2.2.2.4-2 Arrangement of shear walls and braced frames--recommended. Note that the heavy lines indicate shear walls and/or braced frames.

2.2.2.5 Seismic Performance Category E: Sec. 2.2.2.5 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 ft (30 m) and 160 ft (49 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 stringent requirements for detailing the seismic force resisting system as well as the nonstructural components of the building must be stressed. Such requirements are specified in Sec. 2.2.5 and Chapters 5 through 9.

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 for example, the base shear and the distribution of shear throughout the height of a building 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 a 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 2.3.3.1-1 (for period) in Sec. 2.3.3 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 force 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. The internal slabs and columns that resist gravity loads ordinarily 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. 2.2.2.4.3 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 force resisting system (no matter how small), the detailing for ductility must be consistent with the values given in Table 2.2.2. In the example of the flat-slab warehouse, the connections can still carry the design gravity loadings if they satisfy the requirements of Sec. 6.3.3.

2.2.3 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.

2.2.3.1 Plan Irregularity: Sec. 2.2.3.1 indicates, by reference to Table 2.2.3.1, 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. Cracking 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 force 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 2.3.5.1.

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 crucifix form would be classified as an irregular configuration. The response of the wings of this type of building is generally different from 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 C2.2.3.1.

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."

2.2.3.2 Vertical Irregularity: Sec. 2.2.3.2 indicates, by reference to Table 2.2.3.2, 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 Sec. 2.3.

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 C2.2.3.2.

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. 2.2.5.2.4 when there is considerable overstrength of the "weak" story.

2.2.4 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 (Sec. 2.3).

2. Modal analysis procedure with one degree of freedom per floor in the direction being considered (Sec. 2.4).

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.

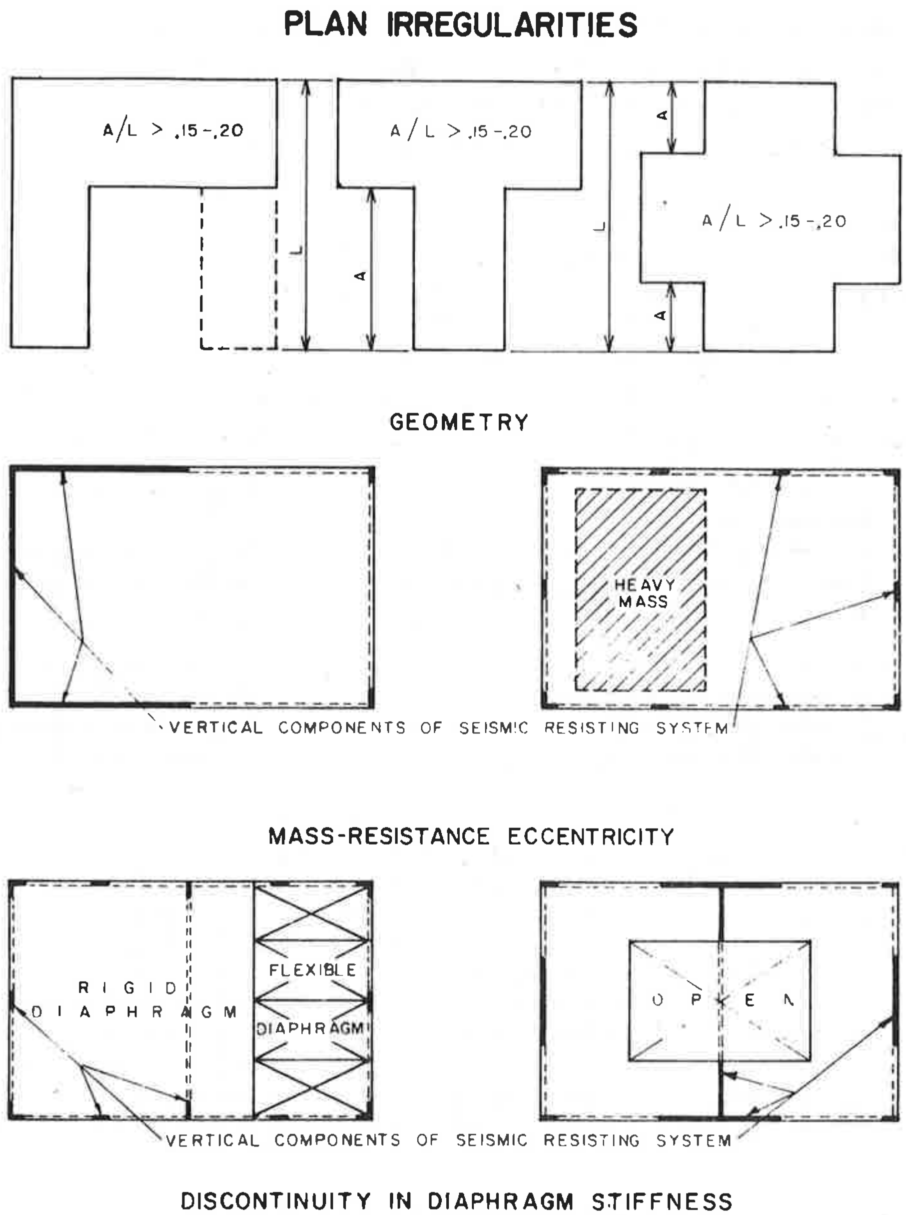


FIGURE C2.2.3.1 Building plan irregularities.

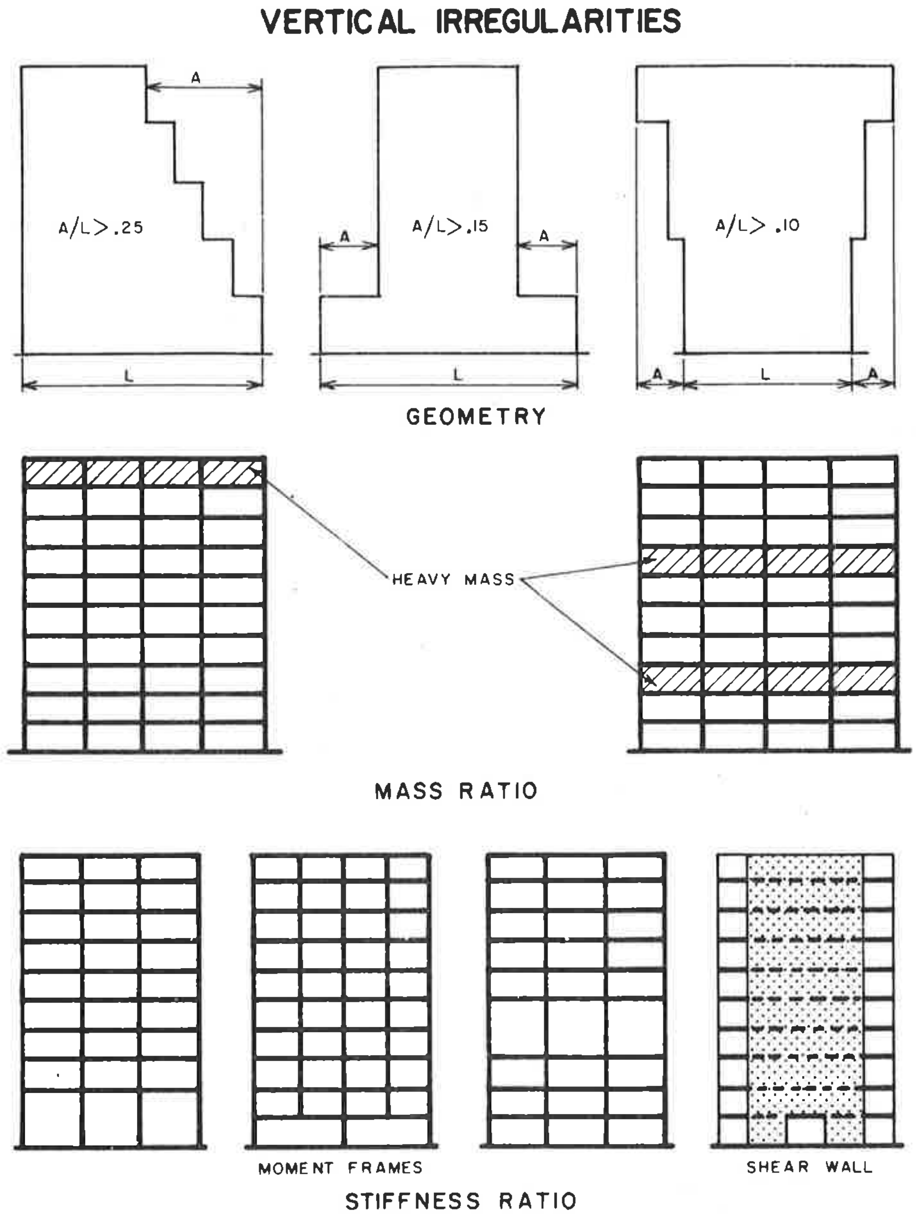


FIGURE C2.2.3.2 Building elevation irregularities.

Each procedure becomes more rigorous if effects of soil-structure interaction are considered, either as presented in Sec. 2.5 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).

The equivalent lateral force (ELF) procedure specified in Sec. 2.3 is similar in its basic concept to SEAOC recommendations in 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 Sec. 2.4 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 Sec. 2.3 and the modal analysis procedure of Sec. 2.4 are both based on the approximation that the effects of yielding can be adequately accounted for by linear analysis of the seismic force 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. 2.3.4. 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. 2.2.3.1) 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 Sec. 2.4 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 (Sec. 2.3) and both versions of the modal analysis procedure (the simple version given in Sec. 2.4 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. 2.2.5.4.2.

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:

1. The number and appropriateness of the time-histories of input motion,

2. The practical limitations of mathematical modeling including interacting effects of nonelastic elements,

3. The nonlinear algorithms, and

4. 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 Sec. 2.4.

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 discussed in Sec. 2.2.3.

Neither regular nor irregular buildings in Seismic Performance Category A are required to be analyzed as a whole for seismic forces, but certain minimum requirements are given in Sec. 2.2.5.1. For the higher Seismic Performance Categories, the ELF procedure is the minimum level of analysis except that a more rigorous procedure is required for some Category D or E buildings as identified in Table 2.2.4.3. 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:

1. Buildings with irregular mass and stiffness properties in which case the simple equations for vertical distribution of lateral forces (Eq. 2.3.4-1 and 2.3.4-2) may lead to erroneous results;

2. Buildings (regular or irregular) in which the lateral motions in two orthogonal directions and the torsional motion are strongly coupled; and

3. 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 Sec. 2.3. 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 **and**

   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 Sec. 2.4.

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 Sec. 2.3 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 Sec. 2.3 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 Sec. 2.4 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 Sec. 2.4 should be used:

1. Compute the story shears using the ELF procedure specified in Sec. 2.3.

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. 2.3.4-2 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 Sec. 2.3 by more than 30 percent, the building should be analyzed using the procedure of Sec. 2.4. 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 Sec. 2.4 are not required.

Application of this procedure to these buildings requires far less computational effort than the use of the modal analysis procedure of Sec. 2.4 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.

Section 2.2.4.3 of the *Provisions* requires "special consideration of dynamic characteristics" when:

1. The building is assigned to Seismic Performance Category D or E and

2. The building has one or more of the plan structural irregularities listed in Table 2.2.3.1 and/or

3. The building has a vertical structural irregularity of Type 4 and/or 5 listed in Table 2.2.3.2.

When special dynamic analysis is required and irregularities of the plan type exist, three-dimensional response spectrum analysis typically will be employed; however, the *Provisions* contain no explicit criteria for such an analysis. Thus, the design engineer must extrapolate the requirements of Sec. 2.4. This commentary is intended to provide the analyst who has determined that three-dimensional analysis is necessary with sufficient guidance in the interpretation of the *Provisions.*

The major components of three-dimensional response spectrum analysis include the following steps (additional comments are provided to clarify certain issues that remain ambiguous):

1. Development of a mathematical model of the structure -- This model should include all possible sources of structural deformation and, when appropriate, may incorporate foundation flexibility and the effects of soil-structure interaction. Sources of structural deformation include axial, flexural, and shear deformation in **all** structural members as well as flexural and shear deformations occurring in the beam-column joints of framed structures. Joint flexibility may be modeled by the use of "centerline" dimensions. Typically, in-plane flexibility of floor diaphragms is insignificant and, hence, the diaphragm may be modelled as infinitely rigid in its own plane. This often allows for significant reduction in computational effort. When diaphragm flexibility exists, the building may be modeled using any number of general-purpose finite element analysis programs. Some special-purpose building analysis programs also account for diaphragm flexibility.

2. P-delta effects -- Since many computer programs automatically include *P*-delta effects, these may be incorporated into the three-dimensional analysis. Gravity forces to be used in *P*-delta analysis include all dead load plus life load reduced according to the applicable code. The analyst should run the dynamic analysis with and without the *P*-delta effects so that the magnitude of these effects may be monitored. If drifts with *P*-delta effects are more than about 1.33 times the drifts without *P*-delta effects, the structure is excessively flexible and should be stiffened.

3. Selection of the design basis response spectrum or spectra -- In lieu of a site-specific spectrum, the spectral shape provided by Eq. 2.4.5-3 may be used. In certain cases, the low period and/or the high period ordinates of the response spectrum may be reduced to values less than those given by Eq. 2.4.5-3.

4. Computation of a sufficient number of "modes" of vibration to capture the dynamic response -- Mode shapes may be represented by traditional Eigenvectors or by Ritz vectors (Wilson et al., 1982). In many cases, a given number of Ritz vectors provide greater accuracy than a larger number of Eigenvectors. Ritz vectors also are more computationally efficient in many circumstances. A sufficient number of mode shapes must be used to capture a minimum of 90 percent of the seismic reactive mass of the structure in each of the two principal directions of response. The first principal response direction may be computed as the arctangent of ratios of the X and Y modal base shears for the first mode of response. The second principal direction of response also lies in the horizontal plane and is orthogonal to the first. It is important to note that a well designed structure should have a minimum amount of torsion in the mode shapes associated with the lowest frequencies of the structure (Wilson et al., 1989).

5. Statistical combination of modal response maxima -- Modal response maxima for three-dimensional structural systems should be combined using the complete quadratic combination (CQC) technique (Wilson et al., 1981). Time history analysis has shown CQC analysis to be more reliable than conventional methods such as the square root of the sum of the squares method (SRSS), particularly when modes are closely spaced. A damping value of 5 percent

of critical may be used in the CQC combinations for steel and reinforced concrete buildings. Larger values may be more appropriate for masonry and timber systems. CQC modal combinations should be performed on the structure for **nonconcurrent** applications of the normalized response spectrum along each principal axis of the building. These nonconcurrent loadings will be used to satisfy the orthogonal loading requirements of the *Provisions*.

The requirements of the *Provisions* that may also need to be incorporated into the analysis include the following:

1. Scaling of results of dynamic analysis -- Section 2.4.8 of the *Provisions* requires that dynamic base shears in each of the principal directions be not less than the shear computed according to Sec. 2.3 but with $1.2T_aC_u$ being substituted for T in Eq. 2.3.2.1-1. If the dynamic base shear in either principal direction is greater than the shear computed according to Sec. 2.3, the shear may be reduced to the value given by Eq. 2.3.2.1-1 with T_aC_u being used for T. These requirements generally will produce different scaling factors in each of the principal directions.

2. Orthogonal loading effects -- Section 2.2.5.4.1 of the *Provisions* requires that Category D or E buildings be designed for orthogonal loading conditions. This may be satisfied by applying 100 percent of the load in one principal direction while applying 30 percent of the load in the orthogonal direction. This can be done in one of two ways:

   a. By applying scaled response spectra simultaneously in the two orthogonal directions--one spectrum having 100 percent of its scaled value and the other having 30 percent of its scaled value. This process should be repeated for loadings rotated 90 degrees about the vertical axis. In each analysis, modal combination should be by CQC.

   b. By applying 100 percent of the scaled spectra in one direction and 30 percent of the spectra in the orthogonal direction **nonconcurrently**. The results from each separate analysis then should be combined using the sum of absolute values (although each separate run would be combined using CQC). The process should be repeated for loadings rotated 90 degrees about the vertical axis.

   A method similar to but somewhat more conservative than Method b above would be to apply 100 percent of the scaled spectra in each direction nonconcurrently and then to combine these results using SRSS (Wilson et al., 1989).

3. Accidental torsion -- Section 2.3.5.1 of the *Provisions* requires that seismic torsional moments, M_t, be added to the equivalent static horizontal seismic forces. These moments result from mass centers for each floor plate being displaced 5 percent of the length of the building perpendicular to the direction of motion. Further, for Category C, D and E buildings with torsional irregularities, these effects must be amplified by A_x. In computing A_x, the analyst needs to know the quantities δ_{min}, δ_{max}, and δ_{avg}, where δ_{avg} is simply $(\delta_{min} + \delta_{max})/2$. The analyst should be careful when using this provision with the results of response spectrum analysis since δ_{min} and δ_{max} will always be positive (signs are lost in

CQC or SRSS modal combinations). The analysis can include torsional effects in one of two ways:

a. By physically displacing the masses thereby directly incorporating the effects into the subsequent dynamic analysis.

b. By computing torsional static moments and applying these as an additional load case. This loading will be combined with the results of dynamic analysis with masses located in the original positions.

The second of the two methods is preferred since it is computationally more efficient and since the effects of torsional eccentricity may be more easily monitored. The first method will give (sometimes substantially) different mode shapes and frequencies than analysis with undisplaced mass. For the first type of analysis, the torsional amplification factor, A_x, may not be necessary.

2.2.5 DESIGN, DETAILING REQUIREMENTS, AND STRUCTURAL COMPONENT LOAD EFFECTS: The design and detailing requirements for components of the seismic force resisting system are stated in this section. The combination of load effects is specified in Sec. 2.2.6. Some of the requirements introduced here are not commonly 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:

1. The actual dynamic characteristics of future earthquake motions expected at a building site;

2. The soil-structure-foundation interaction;

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

4. 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 is 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* uses the concept of Seismic Performance Categories (Table 1.4.4), which relates to the coefficient A_v (Sec. 1.4.1) and the Seismic Hazard Exposure Group (Sec. 1.4.3).

2.2.5.1 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 this section. 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 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.

2.2.5.1.1 Connections: 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. 2.2.5.1.1 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.

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 Sec. 2.2, 2.3, and 2.4 (see Sec. 2.2.5.4.2).

2.2.5.1.2 Anchorage of Concrete or Masonry Walls: 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 $1{,}000A_v$ pounds per linear foot (plf) or 14,600 times A_v Newtons per meter (N/m). 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.

2.2.5.1.3 Anchorage of Nonstructural Systems: Anchorage of nonstructural systems and components of buildings is required when prescribed in Chapter 3.

2.2.5.2 Seismic Performance Category B: Category B and Category 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 still are quite simple compared to present requirements in areas of high seismicity.

The Category B requirements specifically recognize the need to design diaphragms, provide collector bars, and provide reinforcing around openings. There requirements may seem elementary and obvious but, because they are not specifically covered in many codes, some engineers totally neglect them.

2.2.5.2.1 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.

2.2.5.2.5 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 force 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 force 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. 2.2.1.)

The designer should be particularly aware of the proper selection of R when using only one or two one-bay rigid frames in one direction for resisting seismic loads. A single one-bay frame or a pair of such frames provides little redundancy so the designer may wish to consider a modified (smaller) R to account for a lack of redundancy. As more one-bay frames are added to the system, however, overall system redundancy increases. The increase in redundancy is a function of frame placement and total number of frames.

Redundant characteristics also can be obtained by providing several different types of seismic force 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 force resisting system and not to rely on any system wherein distress in any member may cause progressive or catastrophic collapse.

2.2.5.2.6 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 C2.2.5.2.6. 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 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 C2.2.5.2.6 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.

2.2.5.2.7 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 ft (6 m) 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 static and the elastic deformations.

2.2.5.2.8 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.

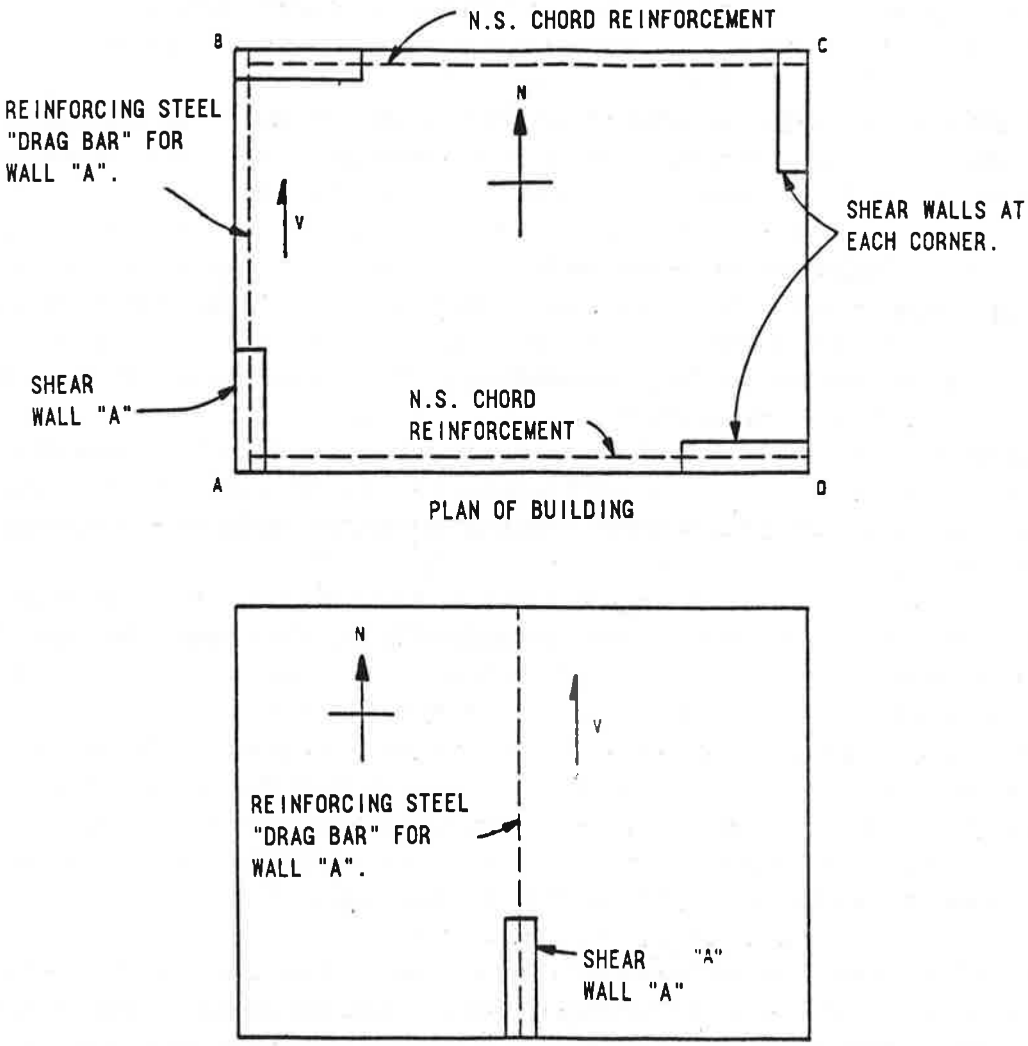


FIGURE 2.2.5.2.6 Collector element used to (a) transfer shears and (b) transfer drag forces from diaphragm to shear wall.

2.2.5.2.9 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. 2.3.2 and 2.3.5 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. 2.3.2 and 2.3.5. 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. 2.3.2 and 2.3.5 is based on analyses of inverted pendulums covering a wide range of practical conditions.

2.2.5.3 Seismic Performance Category C: The material requirements in Chapters 5 through 9 for Category C are somewhat more restrictive than those for Categories A and B. Also, a nominal interconnection between pile caps and caissons is required.

2.2.5.4 Seismic Performance Categories D and E: Category D requirements compare roughly to present design practice in California seismic areas for buildings other than schools and hospitals. 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 and E construction.

Sec. 2.2.5.4.1 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. 2.3.7.

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. 1.4.5). Because of the necessity for reduced risk, height limitations are reduced (see Sec. 2.2.2.5). The specific material provisions include additional requirements and limitations for the design of this building category.

2.2.5.4.1 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 ± 100% of x-direction ± 30% of y-direction
gravity ± 30% of x-direction ± 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.

2.2.5.4.2 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 Sec. 2.3 or 2.4 due to the concentration of inelastic deformations in a weak story. A prohibition on weak story buildings was 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 $2/3r$, modify R and C_d for the building as given by Table 2.2.2 to $\tilde{R}$ and $\tilde{C}_d$ where:

$$\tilde{C}_d = 1 + \frac{C_d - 1}{2}$$

and

$$\tilde{R} = \left(\frac{\tilde{C}_d}{C_d}\right) R$$

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.

2.2.5.4.3 Vertical Seismic Forces 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 $0.5C_a$ which is placed on the dead load (see Sec. 2.2.6). 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. 2.2.6-2 be replaced by Eq. 2.2.6-3. 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 $0.2D$. 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.

2.2.6 COMBINATION OF LOAD EFFECTS: The load combination statements in the 1994 Edition of the *Provisions* combine the effects of structural response to horizontal and vertical ground accelerations. They do not show how to combine the effect of earthquake loading with the effects of other loads. For those combinations, the user is referred to ASCE 7 (Ref. 2-1). The pertinent combinations are:

$$1.2D + 1.0E + 0.5L + 0.2S \qquad \text{(Additive)}$$

and

$$0.9D + 1.0E \qquad \text{(Counteracting)}$$

where D, E, L, and S are, respectively, the dead, earthquake, live, and snow loads.

The design basis expressed in Sec. 2.2.1 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. 2.2.6.-1 and 2.2.6-2.

In Eq. 2.2.6.-1 and 2.2.6-3, a factor of $\mp 0.5C_a$ was placed on the dead load to account for the effects of vertical acceleration. The $0.5C_a$ 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.

The $2R/5$ factor in Eq. 2.2.6-3 and 2.2.6-4 was introduced in the 1991 Edition to better represent the behavior of elements sensitive to overstrength in the remainder of the structural seismic resisting system or in specific other structural components. The particular number was selected to correlate with the $3R_w/8$ factor that had been introduced in Structural Engineers Association of California (SEAoC) recommendations and the *Uniform Building Code*.

2.2.7 DEFLECTION AND DRIFT LIMITS: This section provides procedures for the limitation of story drift. The term "drift" has two connotations:

1. "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).

2. 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 2.2.7 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 except for masonry shear wall buildings.

The drift limits for low-rise structures are relaxed somewhat provided the interior walls, partitions, ceilings, and exterior wall systems have been designed to accommodate story drifts. 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 2.2.7 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. 2.3.7.1.

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, C_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 generally will control, at least in the lower stories.

Due to probable first mode drift contributions and the term C_s being generally conservative at higher values of T or T_a, the Sec. 2.3 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 Sec. 2.4 be used for design even when not required by Sec. 2.2.4.

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 in. (25 mm) plus 1/2 in. (13 mm) for each 10 ft (3 m) of height above 20 ft (6 m) be followed.

2.3 EQUIVALENT LATERAL FORCE PROCEDURE:

2.3.1 GENERAL: This section discusses the equivalent lateral force (ELF) procedure for seismic analysis of buildings.

2.3.2 SEISMIC BASE SHEAR: The heart of the ELF procedure is Eq. 2.3.2-1 for base shear, which gives the total seismic design force, V, in terms of two factors: a seismic response coefficient, C_s, and the total gravity load of the building, W. Equations 2.3.2.1-1 and 2.3.2.1-2 give the coefficient C_s, which defines the design spectrum. This spectrum is discussed more fully in Sec. 1.4.2 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 permanent and movable partitions and 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 (1400 Pa) are not considered. 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 base shear formula and the various factors contained therein were arrived at as explained below.

Elastic Acceleration Response Spectra: See Sec. 1.4.2 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.

Elastic Design Spectra: As described in Sec. 1.4.2, 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. 2.3.2.1-1.

Among the reasons for designing long period buildings more conservatively are the following:

1. 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.

2. 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.

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

Response Modification Factor: The factor R in the denominator of Eq. 2.3.2.1-1 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 2.2.2 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 2.2.2 is explained in the Sec. 2.2.1.

In establishing Eq. 2.3.2.1-1 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* 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. 2.2.7) and by providing special design and detailing requirements (Sec. 2.2.5) and materials limitations (Chapters 5 through 9).

2.3.3 PERIOD DETERMINATION: In the denominator of Eq. 2.3.2.1-1, 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 2.3.3.1-1 and 2.3.3.1-2 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_u.

The coefficient C_u 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 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 2.3.3.1-1 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 C2.3.3-1 and C2.3.3-2.

In these figures, Eq. 2.3.3.1-1 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 $T = 0.1N$ (Eq. 2.3.3.1-2) is an approximation for low to moderate height frames that has been long in use.

As an exception to Eq. 2.3.3.1-1, these provisions allow the calculated fundamental period of vibration, T, of the seismic force resisting system to be used in calculating the base shear. However, the period, T, used may not exceed $C_u T_a$ with T_a determined from Eq. 2.3.3.1-1.

For exceptionally stiff or light buildings, the calculated T for the seismic force resisting system may be significantly shorter than T_a calculated by Eq. 2.3.3.1-1. For such buildings, it is recommended that the period value T be used in lieu of T_a for calculating the seismic response coefficient, C_s.

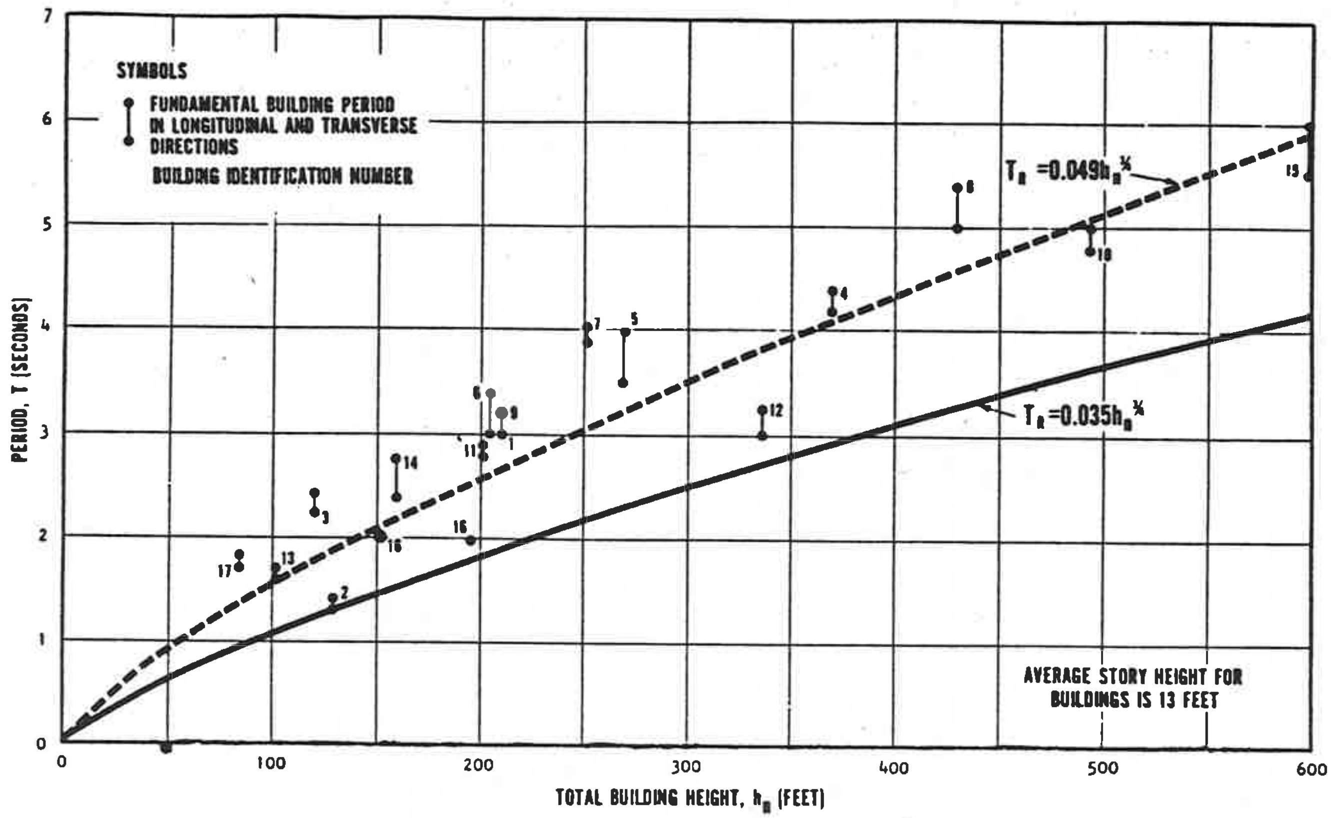


FIGURE C2.3.3-1 Periods computed from accelerograph records during the 1971 San Fernando earthquake--steel frames. The equation $T_R = 0.035h_n^{3/4}$ is intended to be a conservative estimate. The mean value estimate is $T_R = 0.049h_n^{3/4}$. The identification numbers, names, and addresses of the buildings considered are as follows: (1) KB Valley Center, 15910 Ventura; (2) Jet Propulsion Lab Administration Building 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.

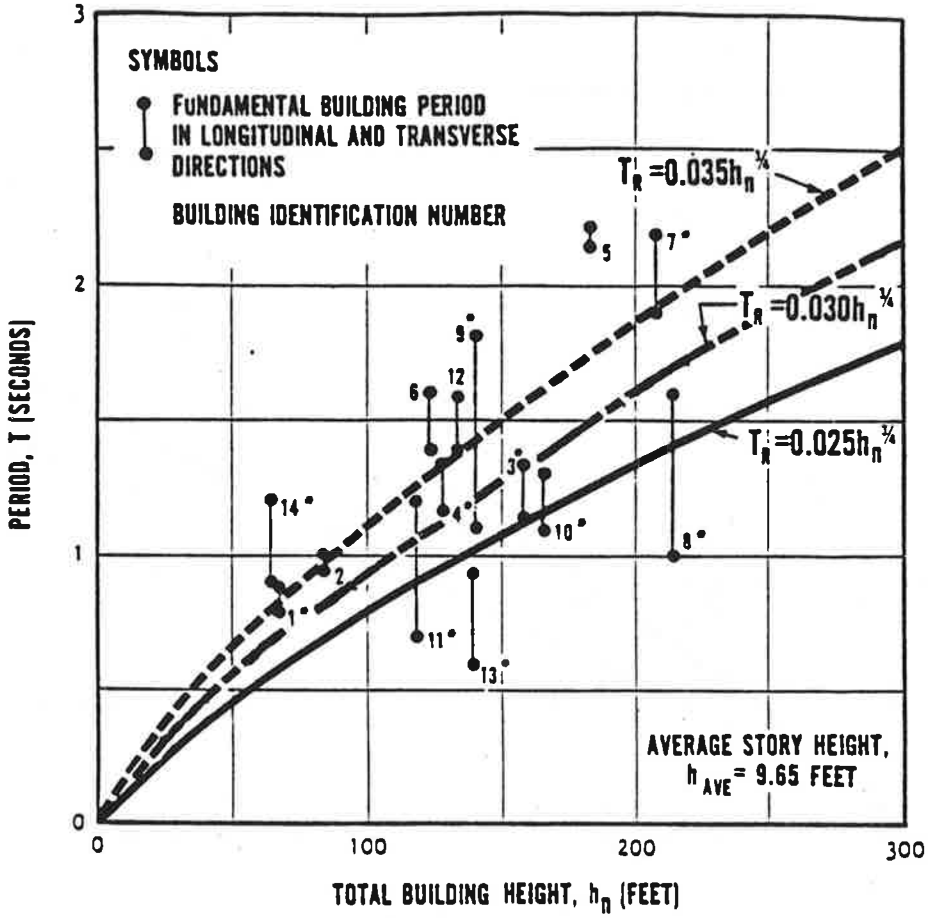


FIGURE C2.3.2-2 Periods computed from accelerograph records during the 1971 San Fernando earthquake--reinforced concrete frames. The equation $T_R = 0.030h_n^{3/4}$ is intended to be a conservative estimate. The mean value estimate is $T_R = 0.035h_n^{3/4}$. The identification numbers, names, and addresses 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.

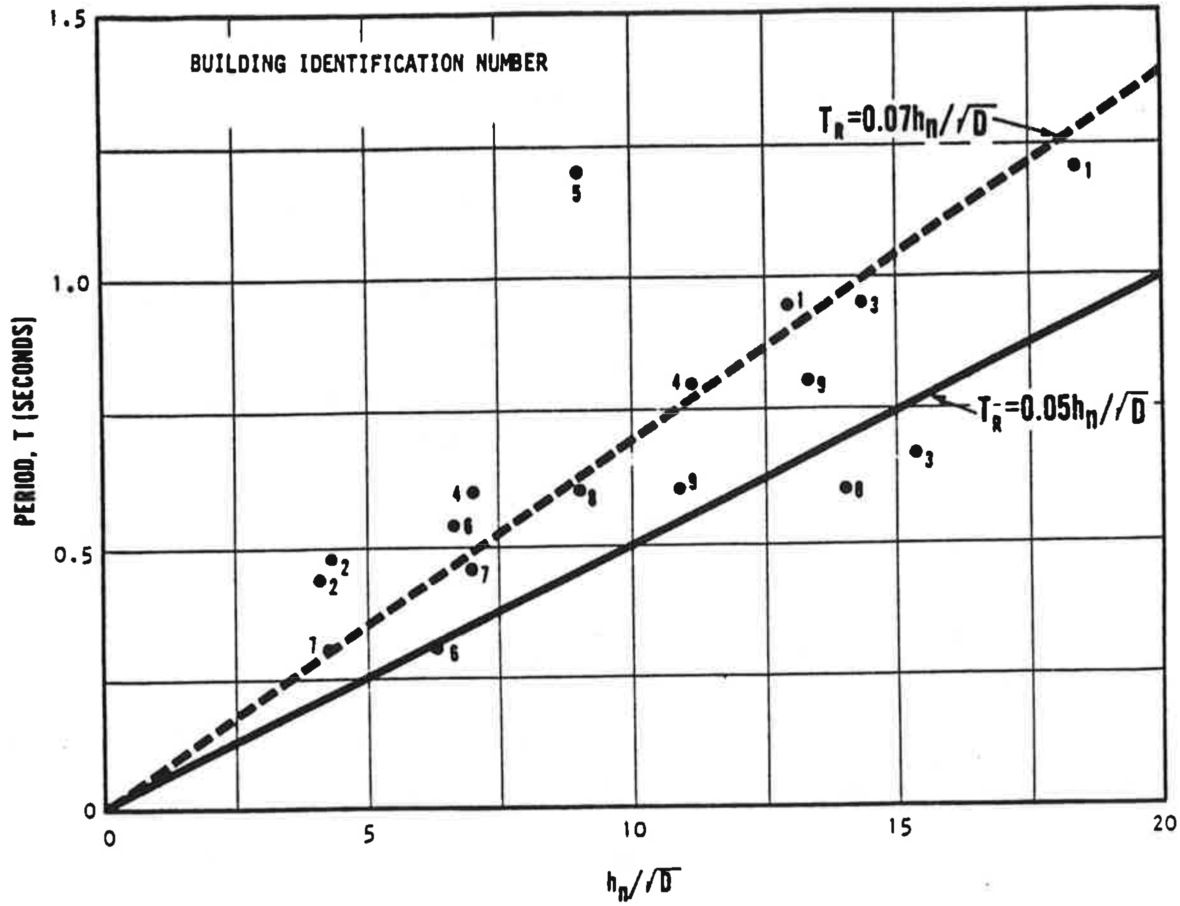


FIGURE C2.3.3-3 Periods computed from accelerograph records during the 1971 San Fernando earthquake--reinforced concrete shear wall buildings. **The equation $T_R = 0.5h_n/\sqrt{D}$ is intended to be a conservative estimate for all buildings other than steel frames and reinforced concrete frames. The mean value estimate is $T_R = 0.07h_n\sqrt{D}$. 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.**

The fundamental period of vibration of the seismic force 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):

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} F_i \delta_i}} \tag{C2.3.3}$$

where:

F_i = the seismic lateral force at Level i,

w_i = the gravity load assigned in Level i,

δ_i = 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 seismic response 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 $C_u T_a$ is imposed as a safeguard.

2.3.4 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. 2.2.3). 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. 2.3.4-2 is much improved by the procedure described in the last portion of Sec. 2.2.4 of this commentary. The lateral force at each level, x, due to response in the first (fundamental) natural mode of vibration is:

$$f_{x1} = V_1 \left(\frac{w_x \phi_{x1}}{\sum_{i=1}^{n} w_i \phi_{i1}} \right) \tag{C2.3.4}$$

where:

V_1 = the contribution of this mode to the base shear,

w_i = the weight lumped at the ith level, and

ϕ_i = the amplitude of the first mode at the i^{th} level.

This is the same as Eq. 2.4.6-2 in Sec. 2.4 of the *Provisions*, but it is specialized for the first mode. If V_1 is replaced by the total base shear, V, this equation becomes identical to Eq. 2.3.4-2 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.

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. 2.3.4-2 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. 2.3.4-2 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.

2.3.5 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 (Figure C2.3.5). 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.

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

1. 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.

2. 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. 2.3.5 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).

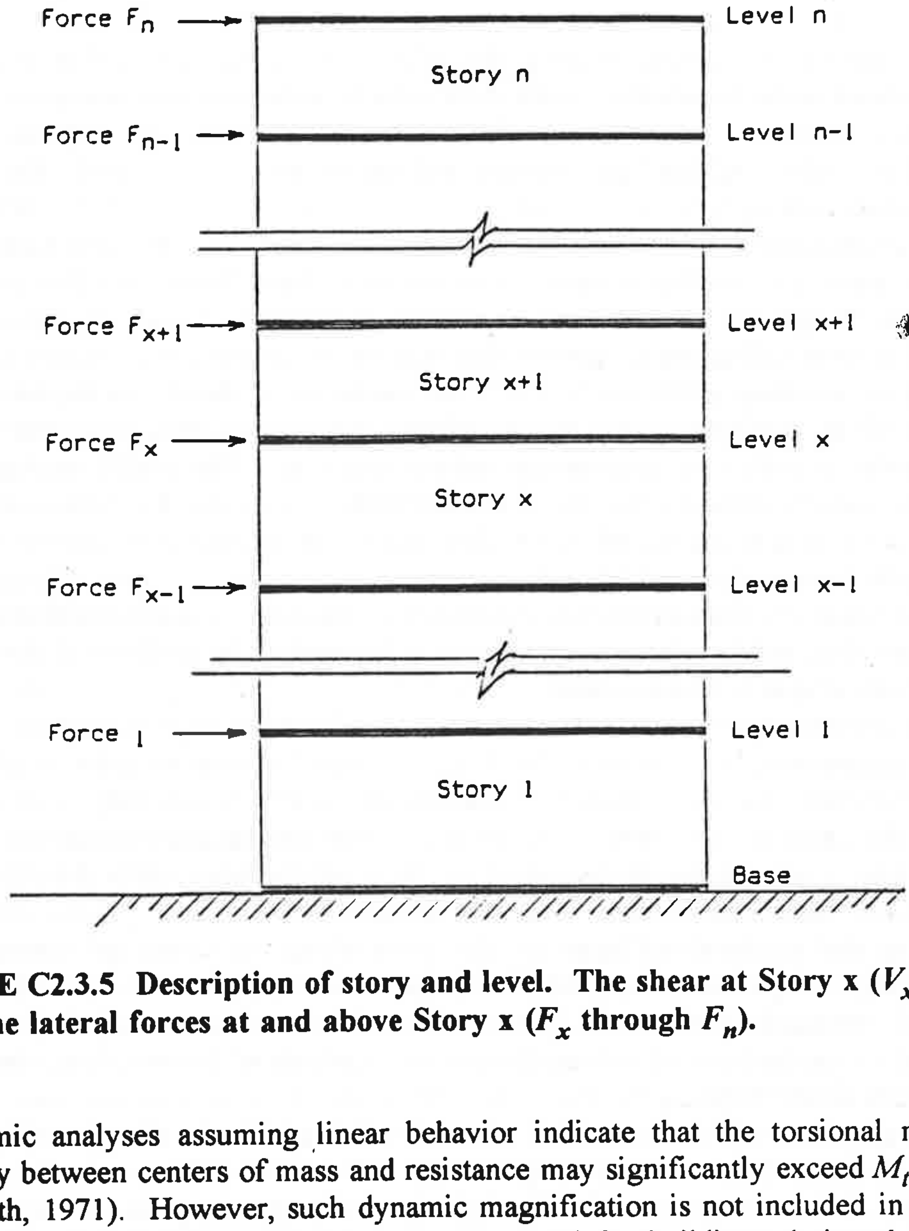


FIGURE C2.3.5 Description of story and level. The shear at Story x (V_x) is the sum of all the lateral forces at and above Story x (F_x through F_n).

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):

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

2. 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. 2.3.5.1) was introduced in the 1988 Edition as an attempt to account for some of these problems in a controlled and rational way.

The way in which the story shears and the effects of torsional moments are distributed to the vertical elements of the seismic force 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.

2.3.6 OVERTURNING: This section requires that the building be designed to resist overturning moments statically consistent with the design story shears, except for reduction factor τ in Eq. 2.3.6. There are several reasons for reducing the statically computed overturning moments:

1. The distribution of design story shears over height computed from the lateral forces of Sec. 2.3.2 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.

2. It is intended that the design shear envelope, which is based on the simple distribution of forces specified in Sec. 2.3.4, 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.

3. Under the action of overturning moments, one edge of the foundation may lift from the ground for short durations of time. Such behavior leads to substantial reduction in the 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 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 τ 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. 2.3.6 with τ = 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 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. 2.3.6, 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.

2.3.7 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 2.2.2. The elastic analysis is to be made for the seismic force 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 force 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. 2.3.4. 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. 2.2.7, the deflections, δ_x, may be calculated as indicated above for the seismic force resisting system and design forces corresponding to the fundamental period of the building, T (calculated without the limit $T \leq C_u T_a$ specified in Sec. 2.3.3), may be used. The same model of the seismic force 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. 2.3.7.2 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. 2.3.4 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. 2.3.7.2-1. 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 $1 + \theta + \theta^2 + \theta^3$ ..., 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 is applicable to planar structures and, with some extension, to three-dimensional structures. Methods exist for incorporating two- and three-dimensional *P*-delta effects into computer analyses that do not explicitly include such effects (Rutenburg, 1982). Many programs explicitly include *P*-delta effects. A mathematical description of the method employed by several popular programs is given by Wilson and Habibullah (1987).

The *P*-delta procedure cited above effectively checks the static stability of a structure based on its initial stiffness. Since the inception of this procedure with ATC 3-06, however, there has been some debate regarding its accuracy. This debate stems from the intuitive notion that the structure's secant stiffness would more accurately represent inelastic *P*-delta effects. Given the additional uncertainty of the effect of dynamic response on *P*-delta behavior and the (apparent) observation that instability-related failures rarely occur in real buildings, the *P*-delta provisions remained as originally written until revised for the 1991 Edition.

There was increasing evidence that the use of inelastic stiffness in determining *theoretical* *P*-delta response is unconservative. Given a study carried out by Bernal (1987), it was argued that *P*-delta amplifiers should be based on secant stiffness and that, in other words, the C_d term in Eq. 2.3.7.2-1 should be deleted. However, since Bernal's study was based on the inelastic response of single-degree-of-freedom elastic-perfectly plastic systems, significant uncertainties existed regarding the extrapolation of the concepts to the complex hysteretic behavior of multi-degree-of-freedom systems.

Another problem with accepting a *P*-delta procedure based on secant stiffness was that design forces would be greatly increased. For example, consider an ordinary moment frame of steel with a C_d of 4.0 and an elastic stability coefficient θ of 0.15. The amplifier for this structure would be 1.0/0.85 = 1.18 according to the 1988 Edition of the *Provisions*. If the *P*-delta effects were based on secant stiffness, however, the stability coefficient would increase to 0.60 and the amplifier would become 1.0/0.4 = 2.50. (Note that the 0.9 in the numerator of the amplifier equation in the 1988 Edition was dropped for this comparison.) This example illustrates that there could be an extreme impact on the provisions if a change was implemented that incorporated *P*-delta amplifiers based on static secant stiffness response.

There was, however, some justification for retaining the *P*-delta amplifier as based on elastic stiffness. This justification was the apparent lack of stability-related failures. The reasons for the lack of observed failures included:

1. Many structures display strength well above the strength implied by code-level design forces (see Figure C2.1.1). This overstrength likely protects structures from stability-related failures.

2. The likelihood of a stability failure decreases with increased intensity of expected ground-shaking. This is due to the fact that the stiffness of most buildings designed for extreme ground motion is significantly greater than the stiffness of the same building designed for lower intensity shaking or for wind. Since damaging low-intensity earthquakes are somewhat rare, there would be little observable damage.

Due to the lack of stability-related failures, therefore, the requirements of the 1988 Edition of the *Provisions* regarding P-delta amplifiers remain in the 1991 and 1994 Editions with the exception that the 0.90 factor in the numerator of the amplifier has been deleted. This factor originally was used to create a transition from cases where P-delta effects need not be considered ($\theta \leq 0.10$, amplifier = 1.0) to cases where such effects need be considered ($\theta > 1.0$, amplifier > 1.0).

However, the 1991 Edition introduced a requirement that the computed stability coefficient, θ, not exceed 0.25 or $0.5/\beta C_d$, where βC_d is an adjusted ductility demand that takes into account the fact that the seismic strength demand may be somewhat less than the code strength supplied. The adjusted ductility demand is not intended to incorporate overstrength beyond that computed by the means available in Chapters 5 through 9 of the *Provisions*.

The purpose of this requirement is to protect structures from the possibility of stability failures triggered by post-earthquake residual deformation. The danger of such failures is real and may not be eliminated by apparently available overstrength. This is particularly true of structures designed in regions of lower seismicity.

The computation of θ_{max}, which, in turn, is based on βC_d, requires the computation of story strength supply and story strength demand. Story strength demand is simply the seismic design shear for the story under consideration. The story strength supply may be computed as the shear in the story that occurs simultaneously with the attainment of the development of first significant yield of the overall structure. To compute first significant yield, the structure should be loaded with a seismic force pattern similar to that used to compute seismic story strength demand. A simple and conservative procedure is to compute the ratio of demand to strength for each member of the seismic force resisting system in a particular story and then use the largest such ratio as β. For a structure otherwise in conformance with the *Provisions*, β = 1.0 is obviously conservative.

The principal reason for inclusion of β is to allow for a more equitable analysis of those structures in which substantial extra strength is provided, whether as a result of added stiffness for drift control, from code-required wind resistance, or simply a feature of other aspects of the design.

$$\beta = \frac{\text{Story Shear Demand}}{\text{Story Shear Capacity}}$$

is conservatively 1.0 for any design that meets the remainder of the *Provisions*. Some structures inherently possess more strength than required, but instability is not typically a concern for such structures. For many flexible structures, the proportions of the structural members are controlled by the drift requirements rather than the strength requirements; consequently, β is less than 1.0 because the members provided are larger and stronger than required. This has the effect of reducing the inelastic component of total seismic drift and, thus, β is placed as a factor on C_d.

Accurate evaluation of β would require consideration of all pertinent load combinations to find the maximum value of seismic load effect demand to seismic load effect capacity in each and every member. A conservative simplification is to divide the total demand with seismic included by the total capacity; this covers all load combinations in which dead and live effects add to seismic. If a member is controlled by a load combination where dead load counteracts seismic, to be correctly computed, the ratio β must be based only on the seismic component, not the total; note that the vertical load P in the P-delta computation would be less in such a circumstance and, therefore, θ would be less. The importance of the counteracting load combination does have to be considered, but it rarely controls instability.

2.4 MODAL ANALYSIS PROCEDURE:

2.4.1 GENERAL and 2.4.2 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. 2.3.2.1-1) are simply an expansion 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 Sec. 2.3. Thus, the coefficient C_{sm} in Eq. 2.4.5-1 and the distribution equations, Eq. 2.4.6-1 and 2.4.6-2, are the counterparts of Eq. 2.3.4-1 and 2.3.4-2. This correspondence helps clarify the fact that the simplified modal analysis contained in Sec. 2.4 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 Sec. 2.3.

The modal analysis procedure specified in Sec. 2.4 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 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.

2.4.3 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.

2.4.4 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. 2.4.5-3. 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.

2.4.5 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. 2.4.5-3 using the associated period of the mode, T_m, in addition to the factors C_v 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 soils of Soil Profile Type D, E, or F. For such modes, Eq. 2.4.5-4 is used. Equation 2.4.5-4 gives values ranging from A_a/R for very short periods to $2.5A_a/R$ for $T_m = 0.3$. Comparing these values to the limiting values of C_s of $2.5A_a/R$ for soils with Soil Profile Type D as specified following Eq. 2.4.5-3, it is seen that the use of Eq. 2.4.5-4, 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. 2.4.5-3 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 2.4.5-3 and its limiting values conservatively replace the ascending portion for small periods by a level portion. For soils with Soil Profile Type A, B and C, 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 2.4.5-4 is then a replacement for the spectral shape for soils with Soil Profile Type D, E and F and short periods that is more consistent with spectra for measured accelerations. It was introduced because it was judged unnecessarily conservative to use Eq. 2.4.5-3 for modal analysis in the case of soils with Soil Profile Types D, E, and F. The effective modal gravity load given in Eq. 2.4.5-2 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. 2.4.5-2 gives values of W_m that are independent of how the modes are normalized.

The final equation of this section, Eq. 2.4.5-5, is to be used if a modal period exceeds 4 seconds. It can be seen that Eq. 2.4.5-5 and 2.4.5-3 coincide at $T_m = 4$ seconds so that the effect of using Eq. 2.4.5-5 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 2.4.5-3 decreases as $1/T_{m2/3}$ for reasons discussed in Sec. 2.3.2 of this 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.

2.4.6 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. 2.4.6-1 and 2.4.6-2 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. 2.4.6-1 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. 2.3.4-1 in the ELF procedure.

The modal deflections at each level are specified by Eq. 2.4.6-3. 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. 2.4.8. 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. 2.4.6-4) and will therefore change direction up and down the structure for the higher modes.

2.4.7 MODAL STORY SHEARS AND MOMENTS: This section merely specifies that the forces of Eq. 2.4.6-1 should be used to calculate the shears and moments for each mode under consideration. In essence, the forces from Eq. 2.4.6-1 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. 2.4.5-1.

2.4.8 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). The 1991 and 1994 Editions of the *Provisions* also include the combining of these quantities by the complete quadratic combination (CQC) technique.

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_u.

2.4.9 HORIZONTAL SHEAR DISTRIBUTION AND TORSION: This section requires that the design story shears calculated in Sec. 2.4.8 and the torsional moments prescribed in Sec. 2.3.5 be distributed to the vertical elements of the seismic resisting system as specified in Sec. 2.3.5 and as elaborated on in the corresponding section of this 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. 2.2.3) 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. 2.2.3.1 of this commentary).

2.4.10 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 Sec. 2.3 (see Sec. 2.3.6 of this 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. 2.3.6 of this commentary.

2.4.11 P-DELTA EFFECTS: Section 2.3.7 of this commentary applies to this section. In addition, to obtain the story drifts when using the modal analysis procedure of Sec. 2.4, the story drift for each mode should be independently determined in each story (Sec. 2.4.6). 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.

2.5 SOIL-STRUCTURE INTERACTION EFFECTS:

2.5.1 GENERAL:

Statement of the Problem: Fundamental to the design provisions presented in Sec. 2.3 and 2.4 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 Sec. 2.5 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 accounted for in Sec. 2.5 should not be confused with "site effects," which 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 coefficients employed in Sec. 2.3 and 2.4 and are accounted for only implicitly in Sec. 2.5.

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 Sec. 2.5.

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 C2.5.1-1) 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.

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 C2.5.1-2 and 2.5.1-3 represent response spectra for the steady-state amplitude of the total shear in the columns of the system considered in Figure C2.5.1-1. 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:

$$\delta_o = \frac{h}{v_s T} \qquad \text{(C2.5.1-1)}$$

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 C2.5.1-2 and 2.5.1-3 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:

$$d_m = \frac{a_m T_o^2}{4\pi^2} \tag{C2.5.1-2}$$

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.

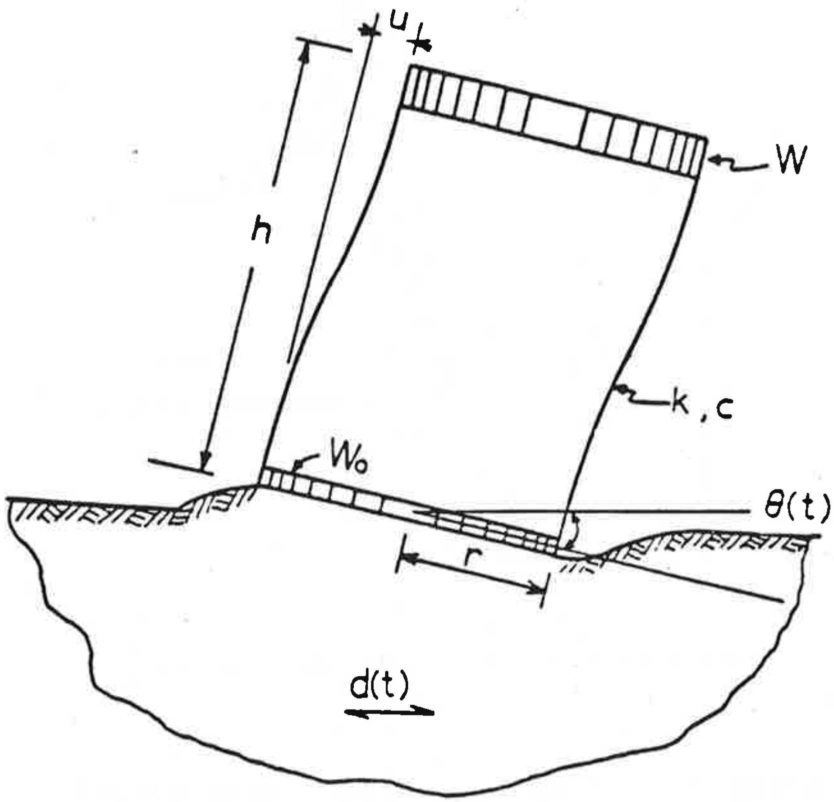


FIGURE C2.5.1-1 Simple system investigated.

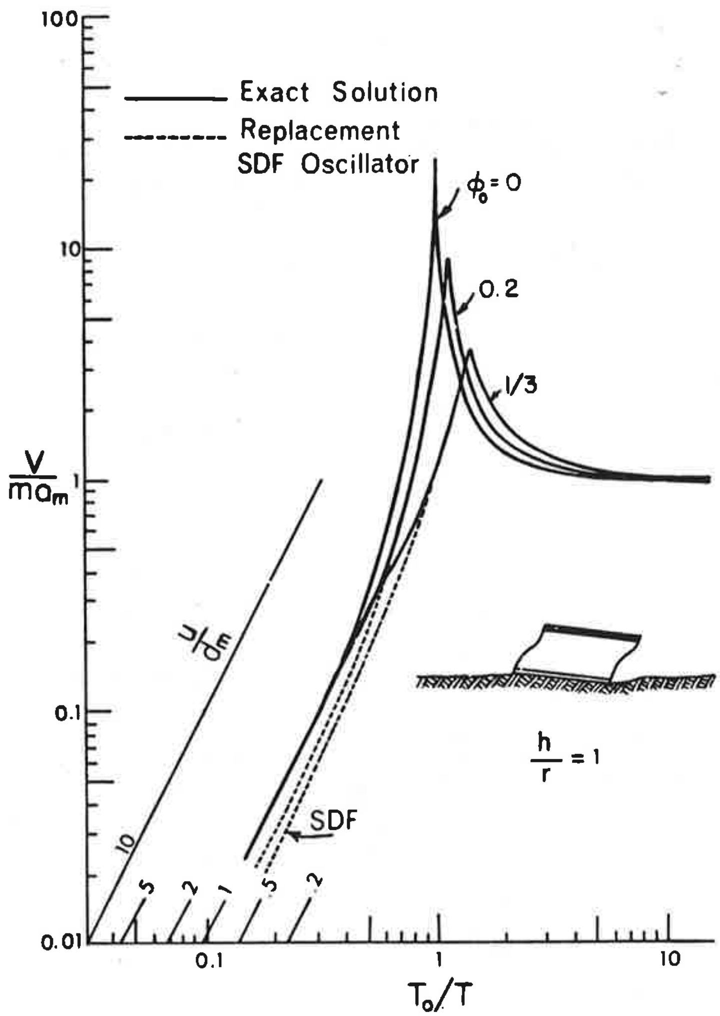


FIGURE C2.5.1-2 Response spectra for systems with *h/r* = 1 (Veletsos and Meek, 1974).

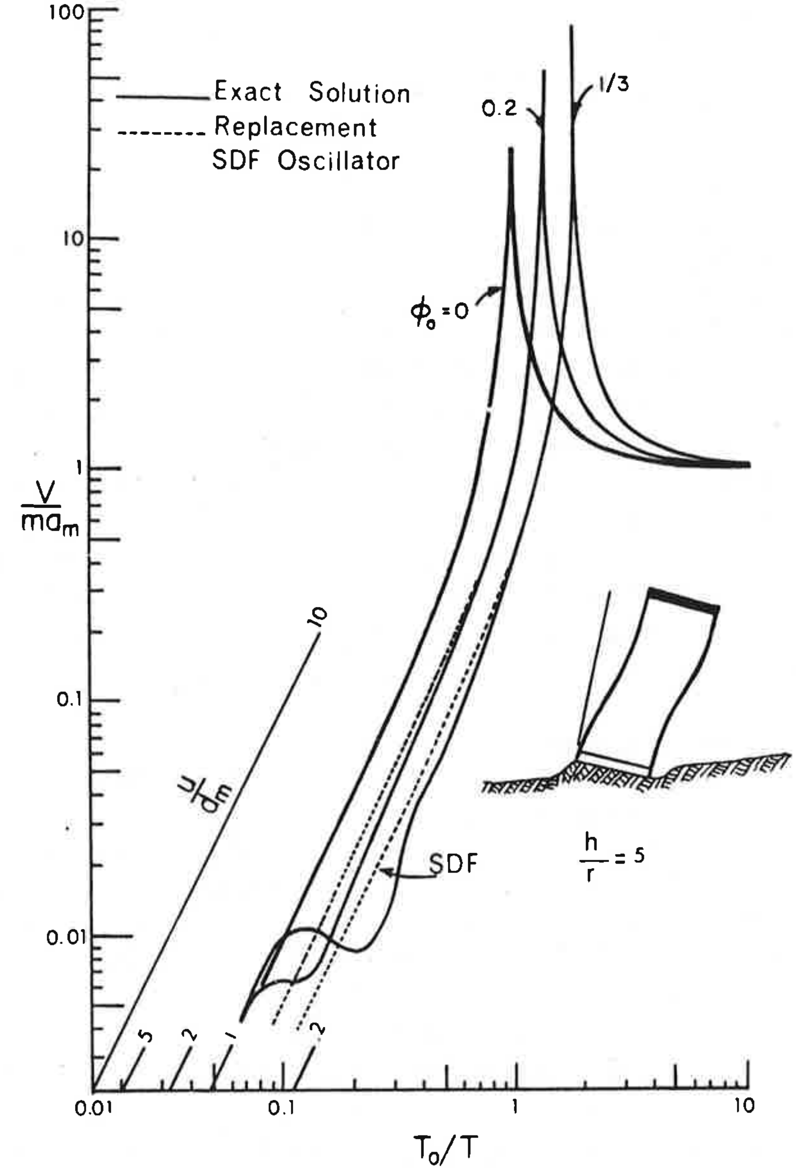


FIGURE C2.5.1-3 Response spectra for systems with *h/r* = 5 (Veletsos and Meek, 1974).

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.

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. 2.5.2.1.1 and 2.5.2.1.2.

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 presented in Sec. 2.5 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 Sec. 2.5. 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 Sec. 2.3 and 2.4, soil-structure interaction will reduce 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 Sec. 2.5.

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 Sec. 2.3 and involves the use of equivalent lateral static forces. The second is an extension of the simplified modal analysis procedure presented in Sec. 2.4. 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 Sec. 2.5. 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.

2.5.2 EQUIVALENT LATERAL FORCE PROCEDURE: This procedure is similar to that used in the older SEAOC recommendations except that it incorporates several improvements (see Sec. 2.3 of this 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).

2.5.2.1 Base Shear: With the effects of soil-structure interaction neglected, the base shear is defined by Eq. 2.3.2:

$$V = C_s W \qquad (2.3.2.1)$$

in which W is the total dead weight of the building and of applicable portions of the design live load (as specified in Sec. 2.3.2) and C_s is the dimensionless seismic response coefficient (as defined by Eq. 2.3.2.1-1). This term 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. 2.3.2.1-1 because a constant value of $\beta = 0.05$ 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. 2.3.2.1 in the form:

$$V = C_s(T,\beta)\overline{W} + C_s(T,\beta)[W - \overline{W}] \qquad (C2.5.2.1\text{-}1)$$

where $\overline{W}$ 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 $\overline{W}$ and W is given below. The first term on the right side of Eq. C2.5.2.1-1 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. C2.5.2.1-1:

$$\tilde{V} = C_s(\tilde{T},\tilde{\beta})\overline{W} + C_s(T,\beta)[W - \overline{W}] \qquad \text{(C2.5.2.1-2)}$$

The value of C_s in the first part 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 part 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. C2.5.2.1-2, it is desirable to rewrite this formula in the same form as Eq. 2.5.2.1-1. Making use of Eq. 2.3.2.1 and rearranging terms, the following expression for the reduction in the base shear is obtained:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\tilde{\beta})\right]\overline{W} \qquad \text{(C2.5.2.1-3)}$$

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:

$$C_s(\tilde{T},\tilde{\beta}) = C_s(\tilde{T},\beta)\left(\frac{\beta}{\tilde{\beta}}\right)^{0.4} \qquad \text{(C2.5.2.1-4)}$$

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. C2.5.2.1-4 in Eq. C2.5.2.1-3 leads to:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\beta)\left(\frac{\beta}{\tilde{\beta}}\right)^{0.4}\right]\overline{W} \qquad \text{(C2.5.2.1-5)}$$

where both values of C_s are now for the damping factor of the rigidly supported system and may be evaluated from Eq. 2.3.2. If the terms 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. C2.5.2.1-5 reduces to Eq. 2.5.2.1-2.

Note that $\tilde{C}_s$ in Eq. 2.5.2.1-2 is smaller than or equal to C_s because Eq. 2.3.2 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. 2.5.2.1.2-1), 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. 2.4.5-2 (Sec. 2.4), 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.

2.5.2.1.1 Effective Building Period: Equation 2.5.2.1.1-1 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 2.5.2.1.1-2 for the structural stiffness, $\bar{k}$, is deduced from the well known expression for the natural period of the fixed-base system:

$$T = 2\pi\sqrt{\left(\frac{1}{g}\right)\left(\frac{\bar{W}}{\bar{k}}\right)} \qquad \text{(C2.5.2.1.1-1)}$$

The effective height, $\bar{h}$, is defined by Eq. 2.5.3.1-2, in which ϕ_{il} has the same meaning as the quantity ϕ_{im} in Eq. 2.4.5-2 (Sec. 2.4) 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 $\bar{h} = 0.7h_n$ 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 $2/3h_n$. 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 **surface** of a homogeneous halfspace, stiffnesses K_y and K_θ are given by:

$$K_y = \frac{8\alpha_y}{(2 - \nu)Gr} \quad \text{(C2.5.2.1.1-2)}$$

and

$$K_\theta = \left[\frac{8\alpha_\theta}{3(1 - \nu)}\right]Gr^3 \quad \text{(C2.5.2.1.1-3)}$$

where r is the radius of the foundation; G is the shear modulus of the halfspace; ν is its Poisson's ratio; and a_y and a_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:

$$G = \frac{\gamma v_s^2}{g} \quad \text{(C2.5.2.1.1-4)}$$

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 a_y and a_θ are unity, and Eq. C2.5.2.1.1-2 and 2.5.2.1.1-3 reduce to:

$$K_y = \frac{8Gr}{2 - \nu} \quad \text{(C2.5.2.1.1-5)}$$

and

$$K_\theta = \frac{8Gr^3}{3(1 - \nu)} \quad \text{(C2.5.2.1.1-6)}$$

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. C2.5.2.1.1-5 and C2.5.2.1.1-6.

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:

$$K_y = \left[\frac{8Gr}{2 - \nu}\right]\left[1 + \left(\frac{2}{3}\right)\left(\frac{d}{r}\right)\right] \qquad \text{(C2.5.2.1.1-7)}$$

and

$$K_\theta = \left[\frac{8Gr^3}{3 - \nu}\right]\left[1 + 2\left(\frac{d}{r}\right)\right] \qquad \text{(C2.5.2.1.1-8)}$$

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. C2.5.2.1.1-7 and C2.5.2.1.1-8. 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. C2.5.2.1.1-7 and C2.5.2.1.1-8 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. C2.5.2.1.1-7:

$$K_y = \left[\frac{8Gr}{2 - \nu}\right]\left[1 + \left(\frac{2}{3}\right)\left(\frac{d}{r}\right)\right]\left[1 + \left(\frac{1}{2}\right)\left(\frac{r}{D_s}\right)\right]\left[1 + \left(\frac{5}{4}\right)\left(\frac{d}{D_s}\right)\right] \qquad \text{(C2.5.2.1.1-9)}$$

Second, using Eq. C2.5.2.1.1-8:

$$K_\theta = \left[\frac{8Gr^3}{3(1 - \nu)}\right]\left[1 + 2\left(\frac{d}{r}\right)\right]\left[1 + \left(\frac{1}{6}\right)\left(\frac{r}{D_s}\right)\right]\left[1 + 0.7\left(\frac{d}{D_s}\right)\right] \qquad \text{(C2.5.2.1.1-10)}$$

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. 2.5.2.1.1-5 is replaced by the quantity:

$$r_a = \sqrt{\frac{A_o}{\pi}} \qquad \text{(C2.5.2.1.1-11)}$$

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. 2.5.2.1.1-6 is replaced by the quantity:

$$r_m = \sqrt[4]{\frac{I_o}{\pi}} \qquad \text{(C2.5.2.1.1-12)}$$

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:

$$K_y = \Sigma k_{yi} \qquad \text{(C2.5.2.1.1-13)}$$

and

$$K_\theta = \Sigma k_{xi} y_i^2 + \Sigma k_{\theta i} \qquad \text{(C2.5.2.1.1-14)}$$

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:

$$k_{yi} = \left(\frac{8G_i r_{ai}}{2 - \nu}\right)\left(\frac{1 + 2/3d_i}{r_{ai}}\right) \qquad \text{(C2.5.2.1.1-15)}$$

$$k_{xi} = \left(\frac{4G_i r_{ai}}{1 - \nu}\right)\left(\frac{1 + 0.4d_i}{r_a}\right) \qquad \text{(C2.5.2.1.1-16)}$$

and

$$k_{\theta i} = \left[\frac{8G_i r_{mi}^3}{2(1 - \nu)}\right]\left[\frac{1 + 2d_i}{r_{mi}}\right] \qquad \text{(C2.5.2.1.1-17)}$$

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; $r_{ai} = \sqrt{A_{o\,i} / \pi}$ is the radius of a circular footing that has the area of the i^{th} footing, A_{oi}; and r_{mi} equals $\sqrt[4]{A_{o\,i}/\pi}$ = 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. C2.5.2.1.1-13 and 2.5.2.1.1-14 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. C2.5.2.1.1-13 and 2.5.2.1.1-14.

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 typically are 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 2.5.2.1.1. 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. C2.5.2.1.1-4. 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:

$$\bar{\sigma}_o = \bar{\sigma}_{os} + \bar{\sigma}_{ob} \qquad \text{(C2.5.2.1.1-18)}$$

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

$$\bar{\sigma}_{os} = \left(\frac{1 + 2K_o}{3}\right)\gamma' x \qquad \text{(C2.5.2.1.1-19)}$$

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:

$$v_{so} = c_1(2.17 - e)(\bar{\sigma})^{0.25} \qquad \text{(C2.5.2.1.1-20)}$$

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:

$$v_{so} = c_2(2.97 - e)(\bar{\sigma})^{0.25} \qquad \text{(C2.5.2.1.1-21)}$$

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 C2.5.2.1.1-21 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:

$$G_o = 1{,}000 S_u \qquad \text{(C2.5.2.1.1-22)}$$

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. C2.5.2.1.1-19 and Eq. C2.5.2.1.1-20 to C2.5.2.1.1.21 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 2.5.2.1.1. 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. C2.5.2.1.1-20 and C2.5.2.1.1-21 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 2.5.2.1.1. 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 2.5.2.1.1 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 $\nu = 0.4$ also will be adequate for practical purposes.

Regarding an alternative approach, note that Eq. 2.5.2.1.1-3 for the period $\tilde{T}$ of buildings supported on mat foundations was deduced from Eq. 2.5.2.1.1-1 by making use of Eq. C2.5.2.1.1-5 and C2.5.2.1.1-6, with Poisson's ratio taken as $\nu = 0.4$ and with the radius r interpreted as r_a in Eq. C2.5.2.1.1-5 and as r_m in Eq. C2.5.2.1.1-6. For a nearly square foundation, for which $r_a \approx r_m \approx r$, Eq. 2.5.2.1.1-3 reduces to:

$$\tilde{T} = T\sqrt{\left[1 + 25\alpha\left(\frac{r\bar{h}}{v_s^2 T^2}\right)\right]\left[1 + \left(\frac{1.12\bar{h}}{r}\right)^2\right]} \qquad \text{(C2.5.2.1.1-23)}$$

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

2.5.2.1.2 Effective Damping: Equation 2.5.2.1.2-1 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 C2.5.1-2 and 2.5.1-3. The result is an expression of the form (Bielak, 1975; Veletsos and Nair, 1975):

$$\tilde{\beta} = \beta_o + \frac{8}{\left(\frac{\tilde{T}}{T}\right)^3} \qquad \text{(C2.5.2.1.2-1)}$$

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 C2.5.2.1.2-1 corresponds to the value of $\beta = 0.05$ used in the development of the response spectra for rigidly supported systems employed in Sec. 2.3.

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 2.5.2.1.2 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 $\Delta W_s/W_s = 1$. 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 2.5.2.1.2 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 h/L_o F 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. 2.5.2.1.1-5). On the other hand, for structures with $h/L_o \geq 1$, 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. 2.5.2.1.1-6).

Subject to the qualifications noted in the following section, the curves in Figure 2.5.2.1.2 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.

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 Sec. 2.5 and in the provisions for rigidly supported structures presented in Sec. 2.3 and 2.4.

Equations 2.5.2.1.2-1 and C2.5.2.1.2-1, in combination with the information presented in Figure 2.5.2.1.2, 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. 2.5.2.1.2-1 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 2.5.2.1.2.

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,

$$T_s = \frac{4D_s}{v_s} \tag{C2.5.2.1.2-2}$$

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 2.5.2.1.2 are applicable only when:

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} \geq 1 \tag{C2.5.2.1.2-3}$$

For

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} < 1 \tag{C2.5.2.1.2-4}$$

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. 2.5.2.1.2-4.

For $T_s/\tilde{T} = 1$, Eq. 2.5.2.1.2-4 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 $\tilde{\beta} = \beta = 0.05$.

2.5.2.2 Vertical Distribution of Seismic Forces and 2.5.2.3 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.

The above procedure is applicable to planar structures and, with some extension, to three-dimensional structures. Methods exist for incorporating two- and three-dimensional P-delta effects into computer analyses that do not explicable include such effects (Rutenburg, 1982). Many programs explicitly include P-delta effects. A mathematical description of the method employed by several popular programs is given by Wilson and Habibullah (1987).

The P-delta procedure cited above effectively checks the static stability of a structure based on its initial stiffness. Since the inception of this procedure in the ATC 3-06 document, however, there has been some debate regarding its accuracy. This debate reflects the intuitive notion that a structure's secant stiffness would more accurately represent inelastic P-delta effects. Due to the additional uncertainty of the effect of dynamic response on P-delta behavior and on the (apparent) observation that instability-related failures rarely occur in real buildings, the *P*-delta provisions as originally written have remained unchanged until now.

There is increasing evidence, however, that the use of inelastic stiffness in determining theoretical *P*-delta response is unconservative. Based on a study carried out by Bernal (1987), it can be argued that *P*-delta amplifiers should be based on secant stiffness. In other words, the C_d term in Eq. 2.3.7.2-1 of the *Provisions* should be deleted. Since Bernal's study was based on the inelastic dynamic response of single-degree-of-freedom elastic-perfectly plastic systems, significant uncertainties exist in the extrapolation of the concepts to the complex hysteretic behavior of multi-degree-of-freedom systems.

Another problem with accepting a *P*-delta procedure based on secant stiffness is that current design forces would be greatly increased. For example, consider an ordinary moment frame of steel with a C_d of 4.0 and an elastic stability coefficient, θ, of 0.15. The amplifier for this structure would be 1.0/0.85 = 1.18 according to the current provisions. If the *P*-delta effects were based on secant stiffness, however, the stability coefficient would increase to 0.60 and the amplifier would become 1.0/0.4 = 2.50. (Note that the 0.9 in the numerator of the amplifier equation in the 1988 Edition of the *Provisions* has been dropped for this comparison.) From this example, it can be seen that there could be an extreme impact on the provisions if a change was implemented that incorporated *P*-delta amplifiers based on static secant stiffness response.

Nevertheless, there must be some justification for retaining the *P*-delta amplifier as based on elastic stiffness. This justification is the apparent lack of stability-related failures. The reasons for the lack of observed failures are, at a minimum, twofold:

1. Many structures display an overstrength well above the strength implied by code-level design forces (see Figure 2.2.1). This overstrength likely protects structures from stability-related failures.

2. The likelihood of a stability failure decreases with the increased intensity of expected ground-shaking. This is due to the fact that the stiffness of most buildings designed for extreme ground motion is significantly greater than the stiffness of the same building deigned for lower intensity shaking or for wind. Since damaging low-intensity earthquakes are somewhat rare, there would be little observable damage.

Due to the lack of stability-related failures, therefore, the 1991 Edition of the *Provisions* regarding *P*-delta amplifiers has remained unchanged from the 1988 Edition with the exception that the 0.90 factor in the numerator of the amplifier has been deleted. This factor originally was used to create a transition from cases where *P*-delta effects need not be considered ($\theta > 1.0$, amplifier > 1.0).

Aside from the amplifier, however, the 1991 Edition of the *Provisions* added a new requirement that the computed stability coefficient, θ, not exceed 0.25 or $0.5/\beta C_d$ where βC_d is an adjusted ductility demand that takes into account the fact that the seismic strength demand may be somewhat less than the code strength supplied. The adjusted ductility demand is **not** intended to incorporate overstrength beyond that computed by the means available in Chapters 5 through 9 of the *Provisions*.

The purpose of this new provision is to protect structures from the possibility of stability-related failures triggered by post-earthquake residual deformation. The danger of such failures is real and may not be eliminated by apparently available overstrength. This is particularly true of structures designed in for regions of lower seismicity.

The computation of θ_{max}, which in turn is based on βC_d, requires the computation of story strength supply and story strength demand. Story strength demand is simply the seismic design shear for the story under consideration. The story strength supply may be computed as the shear in the story that occurs simultaneously with the attainment of the development of first significant yield of the overall structure. To compute first significant yield, the structure should be loaded with a seismic force pattern similar to that used to compute seismic story strength demand. A simple and conservative procedure is to compute the ratio of demand to strength for each member of the seismic force resisting system in a particular story and then use the largest such ratio as β. For a structure otherwise in conformance with the *Provisions*, β = 1.0 is obviously conservative.

The principal reason for inclusion of β is to allow for a more equitable analysis of those structures in which substantial extra strength is provided, whether as a result of adding stiffness for drift control, of code-required wind resistance, or simply of a feature of other aspects of the design.

2.5.3 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:

$$\bar{W} = \bar{W}_1 = \frac{(\Sigma w_i \phi_{i1})^2}{\Sigma w_i \phi_{i1}^2} \tag{C2.5.3}$$

which is the same as Eq. 2.4.5-2, and $\bar{h}$ is computed from Eq. 2.5.3.1-2. 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, k, is obtained from Eq. 2.5.2.1.1-2 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. 2.5.2.1.1-1 (or Eq. 2.5.2.1.2-4 when applicable) by taking $T = T_1$. The effective damping in the first mode, β, is determined from Eq. 2.5.2.1.2-1 (and Eq. 2.5.2.1.2-4 when applicable) in combination with the information given in Figure 2.5.2.1.2. The quantity $\bar{h}$ in the latter figure is computed from Eq. 2.5.3.1-2.

With the values of $\tilde{T}_l$ and $\tilde{\beta}_l$ established, the reduction in the base shear for the first mode, ΔV_l, is computed from Eq. 2.5.2.1-2. The quantities C_s and $\tilde{C}_s$ in this formula should be interpreted as the seismic coefficients corresponding to the periods T_l and $\tilde{T}_l$, respectively; $\tilde{\beta}$ should be taken equal to $\tilde{\beta}_l$; and $\overline{W}$ should be determined from Eq. C2.5.3.

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. 2.5.3.2-1 represents the contribution of the foundation rotation.

2.5.3.3 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. 2.3.5 and the P-delta effects should be evaluated in accordance with the provisions of Sec. 2.3.7.2, using the story shears and drifts determined in Sec. 2.5.3.2.

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_o, 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.

2.6 PROVISIONS FOR SEISMICALLY ISOLATED STRUCTURES: Seismic isolation, commonly referred to as base isolation, is a design concept based on the premise that a structure can be substantially decoupled from potentially damaging earthquake motions. By decoupling the structure from the ground motion, the level of response in the structure can be significantly reduced from the level that would otherwise occur in a conventional fixed-base building. Conversely, seismic isolation permits designing with a reduced level of earthquake load to achieve the same degree of seismic protection and reliability as a conventional fixed-base building.

The potential advantages of seismic isolation and the recent advancements in isolation-system products already have led to the design and construction of over 90 seismically isolated buildings and bridges in the United States. A significant amount of research, development, and application activity has occurred over the past 20 years. The following references provide a summary of some of the work that has been performed: Applied Technology Council (1986, 1993), ASCE Structures Congress (1989, 1991 and 1993), EERI Spectra (1990), Skinner, et al. (1993), U.S. Conference on Earthquake Engineering (1990), and World Conference on Earthquake Engineering (1988, 1992).

In the mid-1980s, the initial application activity identified a need to supplement existing codes with design requirements developed specifically for seismically isolated buildings. Code development work occurred throughout the late 1980s. The status of U.S. seismic isolation design requirements as of August 1993 was as follows:

1. In late 1989, the Structural Engineers Association of California (SEAOC) State Seismology Committee adopted an "Appendix to Chapter 2" of the SEAOC *Blue Book* entitled, "General Requirements for the Design and Construction of Seismic-Isolated Structures." These requirements were submitted to the International Conference of Building Officials (ICBO) and were adopted by the *Uniform Building Code (UBC)* in September 1990. They have been updated on an annual basis since that time.

2. The Building Safety Board (BSB) of California, Office of the State Architect, has adopted *An Acceptable Method for Design and Review of Hospital Buildings Utilizing Base Isolation*. The Northern and Southern Sections of SEAOC completed and submitted to the BSB a revised version of the document; the revision was adopted by the BSB in May 1989. The BSB is currently updating the hospital guidelines to incorporate the SEAOC/UBC criteria developed since 1990.

3. In October 1990, the American Association of State Highway and Transportation Officials (AASHTO) Bridge Committee adopted guide specifications entitled, *Seismic Isolation Design Requirements for Bridge Structures*. These requirements were developed to be compatible with the *1983 AASHTO Guide Specifications for the Seismic Design of Bridges*, which were adopted in October 1990 as the *Standard Design Specifications* by the AASHTO Bridge Committee.

It was decided to use the latest version (1993 approved changes) of the SEAOC/UBC provisions as a basis for the development of the requirements included in the *Provisions*. The only significant changes involved an appropriate conversion to strength design and making the provisions applicable on a national basis.

Rather than addressing a specific method of base isolation, the provisions in Sec. 2.6 provide general design requirements applicable to a wide range of possible seismic isolation systems. Although remaining general, the design requirements rely on mandatory testing of isolation-system hardware to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Some systems may not be capable of demonstrating acceptability by test and, consequently, would not be permitted. In general, acceptable systems will:

1. Remain stable for required design displacements,

2. Provide increasing resistance with increasing displacement,

3. Not degrade under repeated cyclic load, and

4. Have quantifiable engineering parameters (e.g., force-deflection characteristics and damping).

Conceptually, there are four basic types of force-deflection relationships for isolation systems. These idealized relationships are shown in Figure C2.6 with each idealized curve having the same design displacement, D, for the design-level earthquake.

A linear isolation system is represented by Curve A and has the same isolated period for all earthquake load levels. In addition, the force generated in the superstructure is directly proportional to the displacement across the isolation system.

A hardening isolation system is represented by Curve B. This system is soft initially (long effective period) and then stiffens (effective period shortens) as the earthquake load level increases. When the earthquake load level induces displacements in excess of the design displacement in a hardening system, the superstructure is subjected to higher forces and the isolation system to lower displacements than a comparable linear system.

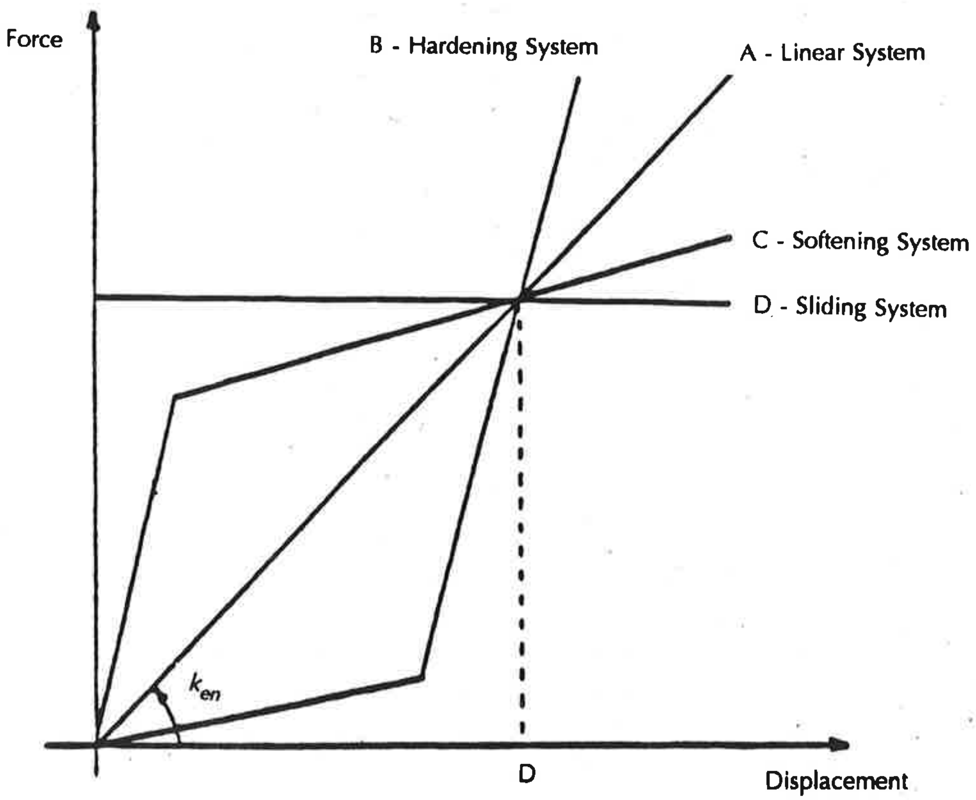


FIGURE C2.6 Idealized force-deflection relationships for isolation systems (stiffness effects of sacrificial wind-restraint systems not shown for clarity).

A softening isolation system is represented by Curve C. This system is stiff initially (short effective period) and softens (effective period lengthens) as the earthquake load level increases. When the earthquake load level induces displacements in excess of the design displacement in a softening system, the superstructure is subjected to lower forces and the isolation system to higher displacements than a comparable linear system.

A sliding isolation system is represented by Curve D. This system is governed by the friction force of the isolation system. Like the softening system, the effective period lengthens as the earthquake load level increases and loads on the superstructure remain constant.

The total system displacement for extreme displacement of the sliding isolation system, after repeated earthquake cycles, is highly dependent on the vibratory characteristics of the ground motion and may exceed the design displacement, D. Consequently, minimum design requirements do not adequately define peak seismic displacement for seismic isolation systems governed solely by friction forces.

2.6.1 GENERAL: The design requirements permit the use of one of three different analysis procedures for determining the design-basis seismic loads. The first procedure uses a simple-lateral-force formula (similar to the lateral-force coefficient now used in conventional building design) to prescribe peak lateral displacement and design force as a function of seismic zone, Soil

Profile Type, proximity to active faults, and isolated-building period and damping. The second and third methods, which are required for geometrically complex or especially flexible buildings, rely on dynamic analysis procedures (either response spectrum or time history) to determine peak response of the isolated building.

The three procedures are based on the same level of seismic input and require a similar level of performance from the building. There are benefits in performing a more complex analysis in that slightly lower design forces and displacements are permitted as the level of analysis becomes more sophisticated. The design requirements for the structural system are based on a severe level of earthquake ground motion, which corresponds approximately to a 500-year return-period event as described by the recommended ground-motion spectra of the *Provisions*. The isolation system, including all connections, supporting structural elements and the "gap," is required to be designed (and tested) for the effects of a 1000-year return-period event. Structural elements above the isolation system, however, are not necessarily required to be designed for the full effects of the 500-year return-period event, but may be designed for slightly reduced loads (i.e., loads reduced by a factor of up to 2.0) if the structural system has sufficient ductility, etc., to respond inelastically without sustaining significant damage. A similar fixed-base structure would be designed for loads reduced by a factor of 8 rather than 2.

Ideally, lateral displacement of an isolated structure will result, predominantly due to the deformations of the isolation system, rather than in distortion of the structure above. Accordingly, the lateral-load-resisting system of the structure above the isolation system should be designed to have sufficient stiffness and strength to avoid large, inelastic displacements. For this reason, the *Provisions* document contains criteria that limit the inelastic response of the structure above the isolation system. Although damage control for the design-basis earthquake is not an explicit objective of the provisions, an isolated structure designed to limit inelastic response of the structural system also will reduce the level of damage that would otherwise occur during an earthquake. In general, isolated structures designed in conformance with the provisions should be able to:

1. Resist minor and moderate levels of earthquake ground motion without damage to structural elements, nonstructural components, or building contents and

2. Resist major levels of earthquake ground motion without failure of the isolation system, without significant damage to structural elements, without extensive damage to nonstructural components, and without major disruption to facility function.

The above performance objectives for isolated structures considerably exceed the performance anticipated for fixed-base structures during moderate and major earthquakes. Table C2.6.1 provides a tabular comparison of the performance expected for isolated and fixed-base structures designed in accordance with the *Provisions*. Loss of function is not included in Table C2.6.1. For certain (fixed-base) facilities, loss of function would not be expected to occur until there is significant structural damage causing closure or restricted access to the building. In other cases, the facility could have only limited or no structural damage but would not be functional as a result of damage to vital nonstructural components and contents. Isolation would be expected to mitigate structural and nonstructural damage and protect the facility against loss of function.

The requirements of Sec. 2.6 provide isolator design displacements, structure-design-shear forces, and other specific requirements for seismically isolated structures. All other design requirements including loads (other than seismic), load combinations, allowable forces and stresses, and horizontal-shear distribution are covered by the applicable sections of the *Provisions* for nonisolated structures.

TABLE C2.6.1 Protection Provided by NEHRP Provisions for Minor, Moderate and Major Levels of Earthquake Ground Motion

Risk Category	Earthquake Ground Motion Level		
	Minor	Moderate	Major
Life safety[a]	F/I	F/I	F/I
Structural damage[b]	F/I	F/I	I
Nonstructural damage[c] (contents damage)	F/I	I	I

[a] Loss of life or serious injury is not expected for fixed-base (F) or isolated (I) buildings.
[b] Significant structural damage is not expected for fixed-base (F) or isolated (I) buildings.
[c] Significant nonstructural (contents) damage is not expected for fixed-base (F) or isolated (I) buildings.

2.6.2 CRITERIA SELECTION: This section delineates the requirements for the use of the equivalent-lateral-force and dynamic methods of analysis and the conditions for developing a site-specific response spectrum. The limitations on the simplified lateral-force design procedure are quite severe at this time. Limitations cover the site location with respect to active faults; soil conditions of the site, the height, regularity and stiffness characteristics of the building; and the characteristics of the isolation system. In fact, the current limitations will necessitate a dynamic analysis for most isolated structures. Additionally, time-history analysis is required to determine the design displacement of the isolation system (and the structure above) for the following isolated structures:

1. Isolated structures with a "nonlinear" isolation system including, but not limited to, isolation systems utilizing friction or sliding surfaces, isolation systems with effective damping values greater than about 30 percent of critical, isolation systems not capable of producing a significant restoring force, and isolation systems that restrain or limit extreme earthquake displacement;

2. Isolated structures with a "nonlinear" structure (above the isolation system) including, but not limited to, structures designed for forces that are less than those specified by the *Provisions* for "essentially-elastic" design; and

3. Isolated structures located on Soil Profile Type E or F sites (i.e., very soft soil).

The restrictions placed on the use of equivalent-lateral-force design procedures effectively require dynamic analysis for virtually all isolated structures. However, lower-bound limits on isolation system design displacements and structural-design forces are specified by the *Provisions* in Sec. 2.6.4 as a percentage of the values prescribed by the equivalent-lateral-force design formulas, even when dynamic analysis is used as the basis for design. These lower-bound limits on key design parameters ensure consistency in the design of isolated structures and serve as a "safety net" against gross under-design. Table C2.6.2 provides a summary of the lower-bound limits on dynamic analysis specified by the *Provisions*.

TABLE C2.6.2 Lower-Bound Limits on Dynamic Analysis Specified as a Percentage of Static-Analysis Design Requirements

Design Parameter	Static Analysis	Dynamic Analysis	
		Response Spectrum	Time History
Design displacement	$D = (gF_vA_vN_sT_I)/(4\pi^2B_I)$	–	–
Total design displacement	$D_T \geq 1.1D$	$\geq 0.9D_T$	$\geq 0.9D_T$
Total maximum displacement	$D_{TM} = M_M D_{TC}$	$\geq 0.8D_{TM}$	$\geq 0.8D_{TM}$
Design shear (at or below isolation system)	$V_b = k_{MAX}D$	$\geq 0.9V_B$	$\geq 0.9V_B$
Design shear ("regular" superstructure)	$V_s = k_{MAX}D/R_I$	$\geq 0.8V_S$	$\geq 0.6V_S$
Design shear ("irregular" superstructure)	$V_s = k_{MAX}DR_I$	$\geq 1.0V_S$	$\geq 0.8V_S$
Drift	$0.010/R_I$	$0.015/R_I$	$0.020/R_I$

A site-specific design spectrum for both the design (500-year) and maximum-capable (1000-year) events have to be developed if the site is within 15 km of an active fault, if the Soil Profile Type is E or F, or if isolated period is greater than 3.0 sec. Lower limits are placed on these site-specific spectra and they must not be less than 80 percent of those given in Sec. 2.6.4.4.

2.6.3 EQUIVALENT-LATERAL-FORCE DESIGN PROCEDURE: The lateral displacements given by Eq. 2.6.3.3.1 approximate the long-period displacements obtained from 5 percent damped response spectra for the soil types defined in Sec. 1.4.2. For Zone 4, these response spectra are considered compatible with a 0.4 EPA at a distance of 15 km from the fault.

2.6.3.3 Minimum-Lateral Displacements: Eq. 2.6.3.3.1 is an estimate of peak displacement in the isolation system. In formulating this equation, some of the terms are similar to those used to define the seismic response coefficient, *C*, in the *Provisions* but are used in either different ways or with different values. The principal differences in use of terms are:

1. D is proportional to T, rather than $T^{2/3}$, as would be implied from the lateral-force equation. The lateral-force equation for conventional buildings was chosen to attenuate more slowly

with period to recognize the need for additional conservatism in tall buildings, which always have longer periods than short or low structures and may experience multimode response. The direct proportionality of displacement to T applies over the portion of each response spectrum where pseudo-relative velocity is constant. It is anticipated that most base-isolation systems will have period values within this range. For very long-period isolation systems (i.e., T greater than 5 sec), Eq. 2.6.3.3.1 will tend to overpredict peak displacement; however, it was felt that the large uncertainty in long-period displacements justified this conservatism for design.

2. Two new terms are contained in Eq. 2.6.3.3.1. The first of these is the coefficient, N_s, which is used to increase the maximum displacement in the regions closer than 15 km from an active fault. Evidence obtained during recent earthquakes suggests that ground displacements may be larger in the region close to the rupture surface. Although definitive data on the extent of this increase are not available, the values of the coefficient, N_s, as given in Table 2.6.3.3.1a are given as recommended increases of both proximity and expected earthquake magnitude. For an maximum capable earthquake event of magnitude 8, N_s is increased from 1 to 1.2 for distances between 10 and 5 km from the fault and is further increased to 1.5 for distances less than 5 km.

3. The maximum displacement, D, given in Eq. 2.6.3.3.1 is estimated from spectral response for an isolated structure with 5 percent damping. The coefficient, B, in Eq. 2.6.3.3.1 is an isolation-system damping-related term that is used to decrease (or increase) the computed displacement when the equivalent damping coefficient of the isolation system is greater (or smaller) than 5 percent of critical damping. Values of coefficient B, for different damping levels, are given in Table 2.6.3.3.1b.

A comparison of values obtained from Eq. 2.6.3.3.1 and those obtained from nonlinear time-history analyses are given in references by Kircher et al., (1988), Lashkari and Kircher, (1993) and Constantinou et al., (1993).

If the true deformational characteristics of the isolation system are not stable or vary with the nature of the load (i.e., rate, amplitude or time dependent), the design displacements should be based on deformational characteristics of the isolation system that give the largest possible deflection (k_{min}) and the design forces should be based on deformational characteristics of the isolation system that give the largest possible force (k_{max}).

If the true deformational characteristics of the isolation system are not stable or vary with the nature of the load (i.e., rate, amplitude or time dependent), the damping level used to determine design displacements and forces should be based on deformational characteristics of the isolation system that represent the minimum amount of energy dissipated during cyclic response at the design level.

The configuration of the isolation system for a seismically isolated building or structure should be selected in such a way as to minimize any eccentricity between the center of mass of the superstructure and the center of rigidity of the isolation system. In this way, the effect of torsion on the displacement of isolation elements will be reduced. As for conventional structures, allowance for accidental eccentricity in both horizontal directions must be considered. Figure C2.6.3.3 defines the terminology used in the provisions and Eq. 2.6.3.3.3 provides a simplified formulae for estimating the response due to torsion in lieu of a more refined analysis. The additional component of displacement due to torsion, as prescribed by Eq. 2.6.3.3.3, increases the design displacement at the corner of the structure by about 15 percent (for a perfectly square building in plan) to about 30 percent (for a very long, rectangular building) if the eccentricity is 5 percent of the maximum plan dimension. Such additional displacement, due to torsion, is appropriate for buildings with an isolation system whose stiffness is uniformly distributed in plan. Isolation systems that have stiffness concentrated toward the perimeter of the building or certain sliding systems that minimize the effects of mass eccentricity will have reduced displacements due to torsion. The *Provisions* permit values of D_T as small as $1.1D$, with proper justification.

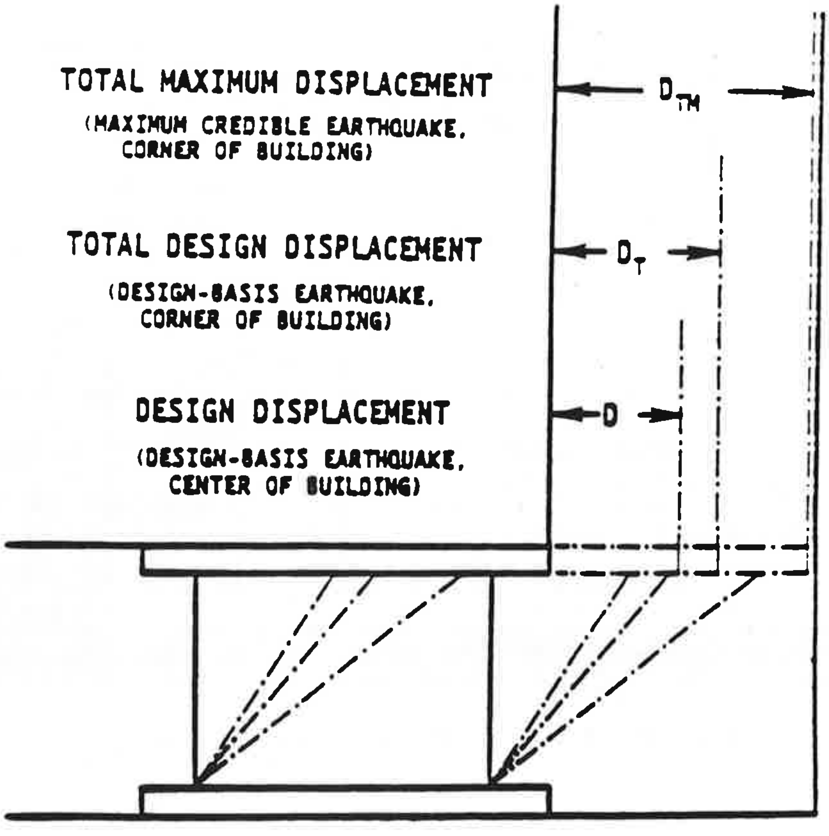


FIGURE C2.6.3.3 Displacement terminology.

Eq. 2.6.3.3.4 provides the total maximum displacement for the lateral-force design procedure where the multiplier, M_M, given in Table 2.6.3.3.4 estimates the relationship between the 1000-year and 500-year design events for different seismic zones.

2.6.3.4 Minimum-Lateral Forces: Figure C2.6.3.4 defines the terminology below and above the isolation system. Eq. 2.6.3.4.1 gives peak seismic shear on all structural components at or below the seismic interface without reduction for ductile response. Eq. 2.6.3.4.2 specifies the peak seismic shear for design of structural systems above the seismic interface. For structures that have appreciable inelastic-deformation capability, this equation includes an effective reduction factor of up to 2 for response beyond the strength-design level.

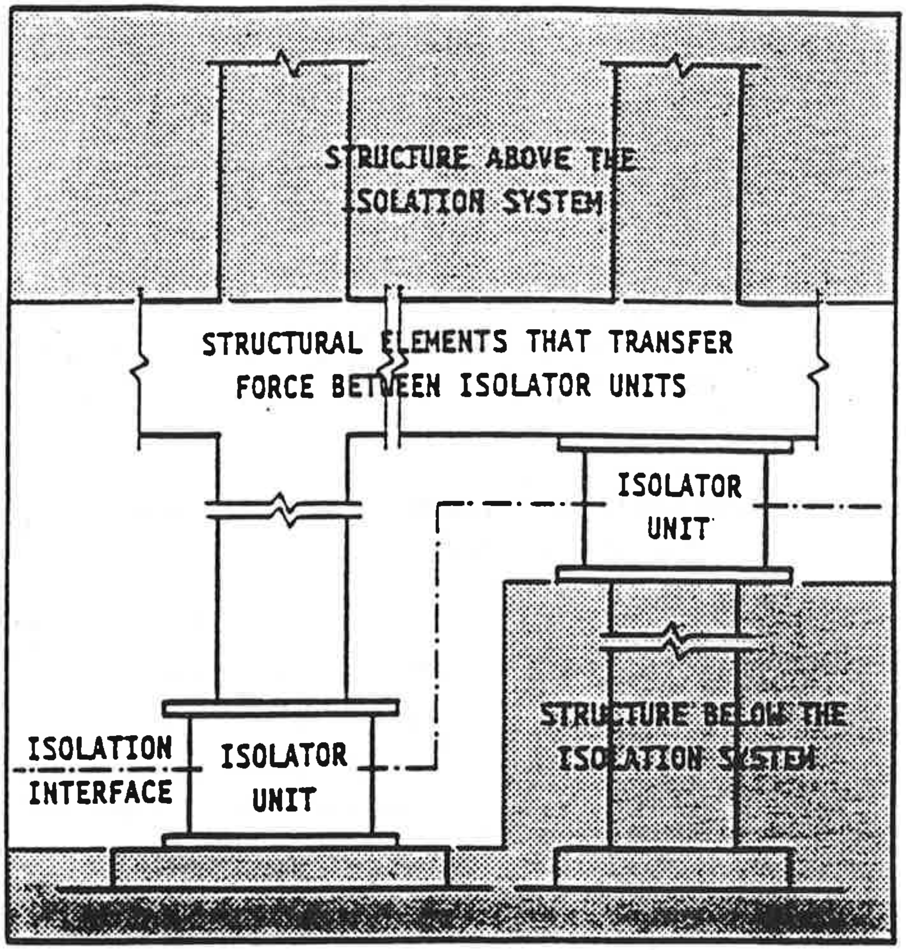


FIGURE C2.6.3.4 Isolation system terminology.

The basis for the reduction factor is that the design of the structural system is based on strength-design procedures. A factor of at least 2 is assumed to exist between the design-force level and the true-yield level of the structural system. An investigation of ten specific buildings indicated that this factor varied between 2 and 5 (Applied Technology Council, 1982). Thus, a reduction factor of 2 is appropriate to ensure that the structural system remains essentially elastic for a 500-year return-period event.

In Sec. 2.6.3.4.3, the limitations given on V_S ensure that there is at least a factor of 1.5 between the nominal yield level of the superstructure and:

1. The yield level of the isolation system;

2. The ultimate capacity of a sacrificial-wind-restraint system which is intended to fail and release the superstructure during significant lateral load; or

3. The static friction level of a sliding system.

These limitations are essential to ensure that the superstructure will not yield prematurely before the isolation system has been activated and significantly displaced.

The design shear force, V_S, specified by the provisions of this section ensures that the structural system of an isolated building will be subjected to significantly less inelastic demands than a conventionally designed structure. Further reduction in V_S, such that the inelastic demand on a seismically isolated structure would be the same as the inelastic demand on a conventionally designed structure, was not considered during development of these requirements but may be considered in the future.

If the level of performance of the isolated structure is desired to be greater than that implicit in these requirements, then the denominator of Eq. 2.6.3.4.2 may be reduced. Decreasing the denominator of Eq. 2.6.3.4.2 will lessen or eliminate inelastic response of the superstructure for the design-basis event.

2.6.3.5 Vertical Distribution of Force: Eq. 2.6.3.5 describes the vertical distribution of lateral force based on an assumed triangular distribution of seismic acceleration over the height of the structure above the isolation interface. References by Button (1993) and Constantinou et al. (1993) provide a good summary of recent work which demonstrates that this vertical distribution of force will always provide a conservative estimate of the distributions obtained from more-detailed-nonlinear analysis studies.

2.6.3.6 Drift Limits: The maximum interstory drift permitted for design of isolated structures varies depending on the method of analysis used, as summarized in Table C2.6.3.6. For comparison, the drift limits prescribed by the *Provisions* for fixed-base structures also are summarized in Table C2.6.3.6.

TABLE C2.6.3.6 Comparison of Drift Limits for Fixed-Base and Isolated Structures

Structure	Static Analysis	Response-Spectrum Analysis	Time-History Analysis
Isolated structures	$0.010/R_I$	$0.015/R_I$	$0.020/R_I$
Fixed-base structures	$0.015/C_d$	$0.015/C_d$	$0.015/C_d$

Drift limits in Table C2.6.3.6 are expressed as a numerical coefficient divided by either R_I for isolated structures or by C_d for fixed-base structures. Drift limits are expressed in terms of $1/R_I$ or $1/C_d$, since they are used to check displacements for "reduced" forces (i.e., elastic-lateral loads reduced either by R_I for isolated structures or by R for fixed-base structures). The C_d term is used throughout the *Provisions* for fixed-base structures to approximate the ratio of actual earthquake response to response calculated for "reduced" forces. Generally, C_d is $\frac{1}{2}$ to $\frac{4}{5}$ the value of R and, consequently, the drift limits for an isolated structure are more conservative than those for a nonisolated structure. Equivalency would be obtained if the drift limits for both fixed-base and isolated structures were divided by their respective *R*-Factors.

2.6.4 DYNAMIC LATERAL RESPONSE PROCEDURE: This section specifies the requirements and limits of a dynamic analysis. The design displacement and force limits on a response-spectrum and time-history analysis are given in Table C2.6.2.

A more-detailed or refined study can be performed in accordance with the analysis procedures described in this section. The intent of this section is to provide analysis procedures which are compatible with the minimum requirements of Sec. 2.6.3. Reasons for performing a more-refined study include:

1. The importance of the building.

2. The need to analyze possible structure/isolation-system interaction when the fixed-base period of the building is greater than one third of the isolated period.

3. The need to explicitly model the deformational characteristics of the lateral-force-resisting system when the structure above the isolation system is irregular.

4. The desirability of using site-specific ground-motion data, especially for soft soil types (Soil Profile Types E or F) or for sites located within 15 kilometers of a major fault.

5. The desirability of explicitly modeling the deformational characteristics of the base-isolation system. This is especially important for systems that have damping characteristics that are amplitude, rather than velocity, dependent, since it is difficult to determine an appropriate value of equivalent viscous damping for these systems.

Additionally, time-history analysis is required to determine the design displacement of the isolation system (and the structure above) for the following isolated structures:

1. Isolated structures with a "nonlinear" isolation system including, but not limited to, isolation systems utilizing friction or sliding surfaces, isolation systems with effective damping values greater than about 30 percent of critical, isolation systems not capable of producing a significant restoring force, and isolation systems that restrain or limit extreme earthquake displacement.

2. Isolated structures with a "nonlinear" structure (above the isolation system) including, but not limited to, structures designed for forces that are less than those specified by the SEAOC/UBC provisions for "essentially-elastic" design.

3. Isolated structures located on Soil Profile Type E or F sites (i.e., very soft soil).

When time-history analysis is used as the basis for design, the design displacement of the isolation system and design forces in elements of the structure above are to be based on the maximum of the results of not less than three separate analyses, each using a different pair of horizontal time histories. Each pair of horizontal time histories is to:

1. Be of a duration consistent with the Design Basis Earthquake (DBE) or the Maximum Capable Earthquake (MCE),

2. Incorporate near-field phenomena, as appropriate, and

3. Have response spectra whose square-root-sum-of-the-squares combination of the two horizontal components equals or exceeds 1.3 times the "target" spectrum at each spectral ordinate.

The average value of seven time histories is a standard required by the nuclear industry and is considered appropriate for nonlinear time-history analysis of seismically isolated structures.

2.6.5 NONSTRUCTURAL COMPONENTS: To accommodate the differential movement between the isolated building and the ground, provision for flexible utility connections should be made. In addition, rigid structures crossing the interface, (i.e., stairs, elevator shafts and walls, should have details to accommodate differential motion at the isolator level without sustaining damage sufficient to threaten life safety.

2.6.6 DETAILED SYSTEM REQUIREMENTS: Environmental conditions that may adversely effect isolation system performance should be thoroughly investigated. Significant research has been conducted on the effects of temperature, aging, etc., on isolation systems since the 1970s in Europe, New Zealand, and the United States.

2.6.6.2.2 Wind forces: Lateral displacement over the depth of the isolator zone resulting from wind loads should be limited to a value similar to that required for other story heights.

2.6.6.2.4 Lateral-restoring force: The isolation system should be configured with a lateral-restoring force sufficient to avoid significant residual displacement as a result of an earthquake, such that the isolated structure will not have a stability problem and be in a condition to survive aftershocks and future earthquakes.

2.6.6.2.6 Vertical-load stability: A recommended factor-of-safety of three for the design of isolation systems for vertical loads was obtained from the NBS Special Publication 577, *Development of a Probability Based Load Criterion for American National Standard A58*. Wherever practical, this factor-of-safety should be achieved.

The application of the factor-of-safety of three will be dependent on the type of isolation system, but should be applied as a well-defined, limiting-stress or -strain value for elastomeric systems. For example, systems with roller bearings may be governed by contact pressure, and for elastomeric bearings the limit will be governed by the tensile strain in the rubber. The required check of stability will ensure that there is sufficient margin prior to any loss of vertical-load support capacity.

2.6.6.2.7 Overturning: The intent of this requirement is to prevent global, structural overturning and overstress of elements due to local uplift. Uplift in a braced frame or shear wall is acceptable, provided the isolation system does not disengage from its horizontal-resisting connection detail. The connection details used in some isolation systems are such that tension is not permitted on the system. If the tension capacity of an isolation system is to be utilized on resisting uplift forces, then component tests should be performed to demonstrate the adequacy of the system on resisting-tension forces at the design displacement.

2.6.6.2.8 Inspection and Replacement: Although most isolation systems will not need to be replaced after an earthquake, it is good practice to provide for inspection and replacement. After an earthquake, the building should be inspected and any damaged elements should be replaced or repaired. It is advised that periodic inspections be made of the isolation system.

2.6.6.2.9 Quality Control: A test and inspection program is necessary for both fabrication and installation of the isolation system. Because base isolation is a developing technology, it may be difficult to reference standards for testing and inspection. Reference can be made to standards for some materials such as elastomeric bearings (ASTM D4014-81 and British standards). Similar standards are required for other isolation systems. Special inspection procedures and load testing to verify manufacturing quality should be developed for each project. The requirements will vary with the type of isolation system used.

2.6.6.3.2 Building Separations: A minimum separation between the isolated structure and a rigid obstruction is required to allow free movement in all lateral directions of the superstructure during an earthquake. Provision should be made for lateral motion greater than the design displacement, since the exact upper limit of displacement cannot be precisely determined.

2.6.9 REQUIRED TESTS OF THE ISOLATION SYSTEM: The design displacements and forces developed from these provisions are predicated on the basis that the deformational characteristics of the base isolation system have been previously defined by a comprehensive set of tests. If a comprehensive amount of test data are not available on a system, then major design alterations in the building may be necessary after the tests are complete. This would result from variations in the isolation-system properties assumed for design and those obtained by test. Therefore, it is advisable that prototype systems be tested during the early phases of design, if sufficient test data is not available on an isolation system.

Typical force-deflection or hysteresis loops are shown in Figure C2.6.9; also included are the definitions of values used in Sec. 2.6.9.3.

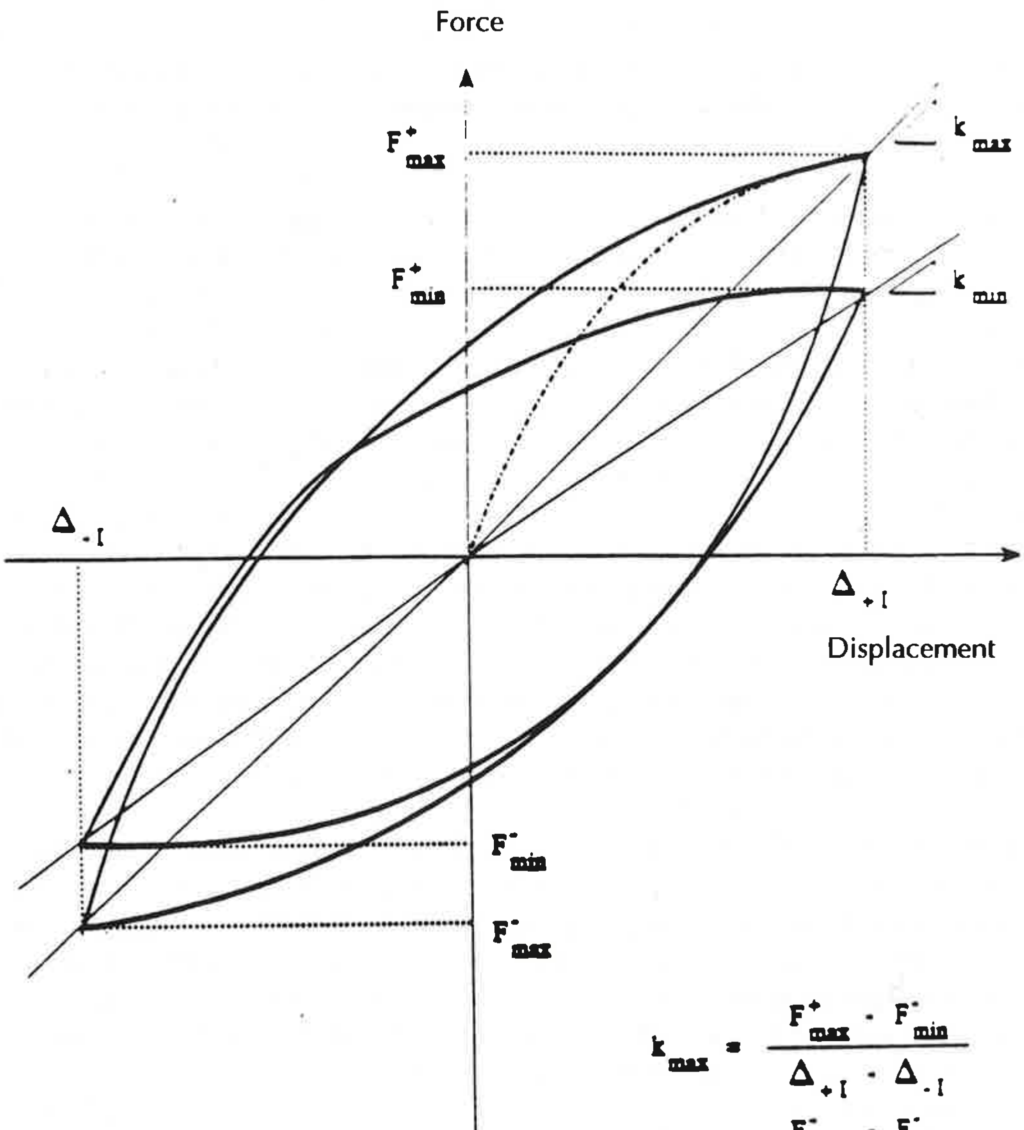
The required sequence of tests will experimentally verify:

1. The assumed stiffness and capacity of the wind-restraining mechanism;

2. The variation in the isolator's deformational characteristics with amplitude and with vertical load, if it is a vertical load-carrying member;

3. The variation in the isolator's deformational characteristics for a realistic number of cycles of loading at the design displacement; and

4. The ability of the system to carry its maximum and minimum vertical loads at the maximum displacement.



$$k_{max} = \frac{F^+_{max} - F^-_{min}}{\Delta_{+I} - \Delta_{-I}}$$

$$k_{min} = \frac{F^-_{min} - F^-_{min}}{\Delta_{+I} - \Delta_{-I}}$$

FIGURE C2.6.9 The effect of stiffness of an isolation bearing.

Force-deflection tests are not required if similarly-sized components have been previously tested using the specified sequence of tests.

Variations in effective stiffness greater than ±15 percent over 3 cycles of loading at a given amplitude, or ±20 percent over the larger number of cycles at the design displacement, would be cause for rejection. The variations in the vertical loads required for tests of isolators which carry vertical, as well as lateral, load are necessary to determine possible variations in the system properties with variations in overturning force. Test set-ups may not be capable of incorporating very low vertical loads because of static instability in the test assembly. Consequently, a compromise may be required to set a limit on the lower vertical load. The appropriate dead loads and overturning forces for the tests are defined as the average loads on a given type and size of isolator for determining design properties and are the absolute maximum and minimum loads for the stability tests.

2.6.9.5.1 Effective Stiffness: The effective stiffness is determined from the hysteresis loops shown in Figure C2.6.9). Stiffness may vary considerably as the test amplitude increases but should be reasonably stable (±15 percent) for more than 3 cycles at a given amplitude.

The intent of these requirements is to ensure that the deformational properties used in design result in the maximum design forces and displacements. For determining design displacement, this means using the lowest damping and effective-stiffness values. For determining design forces, this means using the lowest damping value and the greatest stiffness value.

2.6.9.5.2 Effective Damping: The determination of equivalent viscous damping is reasonably reliable for systems whose damping characteristics are velocity dependent. For systems that have amplitude-dependent, energy-dissipating mechanisms, significant problems arise in determining an equivalent viscous-damping value. Since it is difficult to relate velocity and amplitude-dependent phenomena, it is recommended that when the equivalent-viscous damping assumed for the design of amplitude-dependent, energy-dissipating mechanisms (eg., pure-sliding systems) is greater than 30 percent, then the design-basis force and displacement should be determined by the time-history-analysis method, as specified in Sec. C2.6.2.

2.7 PROVISIONS FOR NONBUILDING STRUCTURES: This section has been added to the 1994 *Provisions* to provide guidance for nonbuilding structures. Previous lateral force provisions were developed primarily for use in building-related applications. However, these building seismic design provisions often have been applied to nonbuilding structures in the absence of definitive guidelines for their design. These new guidelines have been developed from those introduced in the 1988 *Uniform Building Code* to avoid the possible misapplication of guidelines for buildings to nonbuilding structures.

The basic definition of nonbuilding structures stipulates self-supporting structures that carry gravity loads and resist earthquake forces. Further, coverage should be limited to the types of structures that are, or that can be, under the purview of a building official. Certain nonbuilding structures should be exempted from local review such as offshore platforms, electrical transmission towers, dams, and bridges.

Sec. 2.7.2 covers those nonbuilding structures that have structural systems and are therefore building-like. Sec. 2.7.3 covers rigid nonbuilding structures. Sec. 2.7.4 covers grade supported tanks. Sec. 2.7.5 covers all other nonbuilding structures.

In general, R values assigned to nonbuilding structures have been converted from *Uniform Building Code* R_w values and are less than those assigned to buildings. This is because buildings tend to have structural redundancy due to multiple bays and frame lines. Buildings also contain nonstructural elements that give them greater damping and strength when subjected to earthquake motions. The assumed reduced capacity level inherent in values of Table 2.7.5 should not be taken for granted. The structural engineer, therefore, must evaluate the seismic response characteristics of the specific nonbuilding structure to determine if a more conservative R value is appropriate for design.

Hazardous contents are usually defined in occupancy sections of model building codes. The *Uniform Building Code* has cross-referenced these sections in order to provide guidance to building officials in determining hazardous content systems.

REFERENCES

Applied Technology Council. 1982. *An Investigation of the Correlation Between Earthquake Ground Motion and Building Performance*, ATC Report 10. Redwood City, California: ATC

Applied Technology Council. 1986. *Proceedings of a Seminar and Workshop on Base Isolation and Passive Energy Dissipation*, ATC Report 17. Redwood City, California: ATC.

Applied Technology Council. 1993. *Proceedings of Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, ATC 17-1. Redwood City, California: ATC.

American Society of Civil Engineers. 1989. *Seismic Engineering: Research and Practice*. New York City: ASCE.

American Society of Civil Engineers. 1991. *Seismic Engineering: Research and Practice*. New York City: ASCE.

American Society of Civil Engineers. 1993. *Seismic Engineering: Research and Practice*. New York City: ASCE.

Anderson, D. G., and F. E. Richart, Jr. 1976. "Effects of Straining on Shear Modulus of Clays." *Journal of the ASCE Geotechnical Engineering Division* 102 (679):975-987.

Arias, A., and R. Husid. 1962. "Influence of Damping on Earthquake Response of Structures." *Revista del IDIEM* 1 (3). In Spanish.

Ballard, R. F., and F. G. McLean. 1975. "Seismic Field Methods for In Situ Moduli." In *Proceedings of the Conference on In Situ Measurement of Soil Properties*, Vol. I, pp. 121-150. North Carolina State University.

Bernal, D. 1987. "Amplification Factors for Inelastic Dynamic P-delta Effects in Earthquake Analysis." *Earthquake Engineering and Structural Dynamics*, 15:835-881.

Bernal, D. 1987. "Amplification Factors for Inelastic Dynamic *P*-delta Effects in Earthquake Analysis." In *Earthquake Engineering and Structural Dynamics*, Vol. 15, pp. 635-681. New York: John Wiley and Sons.

Bielak, J. 1976. "Modal Analysis for Building-Soil Interaction." *Journal of the ASCE Engineering Mechanics Division* 102 (EM5):771-786.

Bielak, J. 1975. "Dynamic Behavior of Structures with Embedded Foundations." In *Earthquake Engineering and Structural Dynamics*, Vol. 3, pp. 259-274.

Blaney, G. W., I. Kausel, and J. M. Roesset. "Dynamic Stiffness of Piles." In *Proceedings of the Second International Conference on Numerical Methods in Geomechanics*, Vol. 2, pp. 1001-1012. Blacksburg: Virginia Polytechnic Institute and State University.

Chopra, A. K., and J. A. Gutierrez. 1974. "Earthquake Analysis of Multistory Buildings Including Foundation Interaction." *Journal of Earthquake Engineering and Structural Dynamics* 3:65-67.

Clough, R. W. and J. Penzien. 1975. *Dynamics of Structures*. New York: McGraw-Hill.

Constantinou, M. C., C. W. Winters, and D. Theodossiou. 1993. "Evaluation of SEAOC and UBC analysis procedures, Part 2: Flexible superstructure," in *Proceedings of a Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control*, ATC Report 17-1. Redwood City, California: ATC.

Earthquake Engineering Research Institute. 1990. "Seismic isolation: from idea to reality," *Earthquake Spectra Journal* 6:2.

Earthquake Engineering Research Laboratory. 1973-74. *Strong Motion Earthquake Accelerograms*, Vol. II and IV, Parts C-S. Pasadena: EERL.

Elsabee, F., I. Kausel, and J. M. Roesset. 1977. "Dynamic Stiffness of Embedded Foundations." In *Proceedings of the ASCE Second Annual Engineering Mechanics Division Specialty Conference*, pp. 40-43.

Erden, S. M. 1974. Influence of Shape and Embedment on Dynamic Foundation Response. Thesis submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy, University of Massachusetts, Amherst.

Hara, A., T. Ohta, M. Niwa, S. Tanaka, and T. Banno. 1974. "Shear Modulus and Shear Strength of Cohesive Soils." *Journal of the Japanese Society of Soil Mechanics and Foundation Engineering* 14 (3):1-24.

Hardin, B. O., and V. P. Drnevich. 1972. "Shear Modulus and Damping in Soils: Design Equations and Curves." *Journal of the ASCE Soil Mechanics and Foundation Division* 98 (SM7):667-692.

Jennings, P. C., and J. Bielak. 1973. "Dynamics of Building-Soil Interaction." *Bulletin of Seismology Society of America* 63 (1):9-48.

Kausel, E., and J. M. Roesset. 1975. "Dynamic Stiffness of Circular Foundations." *Journal of the ASCE Engineering Mechanics Division* 101 (EM6):771-785.

Kircher, C. A., B. Lashkari, R. L. Mayes, T. E. and Kelly. 1988. "Evaluation of nonlinear response in seismically isolated buildings," in *Proceedings of a Symposium on Seismic, Shock and Vibration Isolation*, ASMA PVP Conference.

Kuribayashi, E., T. Iwasaki, and F. Tatsuoka. 1974. "Effects of Stress Conditions on Dynamic Properties of Sands." *Bulletin of the International Institute of Seismology and Earthquake Engineering* 12:117-130.

Lashkari, B., and C. A. Kircher. 1993. "Evaluation of SEAOC & UBC analysis procedures, part 1: stiff superstructure," in *Proceedings of a Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control*, ATC Report 17-1. Redwood City, California: ATC.

Lashkari, B., and C. A. Kircher. 1990. *Proceedings of Fourth U.S. National Conference on Earthquake Engineering*. Berkeley, California: Earthquake Engineering Research Institute.

Liu, S. C., and L. W. Fagel. 1971. "Earthquake Interaction by Fast Fourier Transform." *Journal of the ASCE Engineering Mechanics Division* 97 (EM4):1223-1237.

Luco, J. E., and R. A. Westmann. 1971. "Dynamic Response of Circular Footings." *Journal of the ASCE Engineering Mechanics Division* 97 (EM5): 1381-1395.

Luco, J. E. 1974. "Impedance Functions for a Rigid Foundation on a Layered Medium." *Nuclear Engineering and Design* (31):204-217.

Murphy, L. M. 1973. *San Fernando California Earthquake of February 9, 1971.* 3 Vol. Washington D.C.: U. S. Department of Commerce, NOAA.

National Bureau of Standards. 1980. *Development of a Probability Based Load Criterion for American National Standards A58*, Special Publication 577. Gaithersburg, Maryland: NBS.

Nair, K., H. Gray, and N. C. Donovan. 1969. *Analysis of Pile Group Behavior.* ASTM Special Technical Publication 44.

Newmark, N. M., J. A. Blume, and K. K. Kapur. 1973. "Seismic Design Spectra for Nuclear Power Plants." *Journal of the ASCE Power Division* 99 (PO 2):873-889.

Newmark, N. M., and E. Rosenblueth. 1971. *Fundamentals of Earthquake Engineering*. New York: Prentice-Hall.

Novak, M. 1974. "Effect of Soil on Structural Response to Wind and Earthquake." In *Earthquake Engineering and Structural Dynamics*, Vol. 3, pp. 79-96.

Novak, M., and Y. O. Beredugo. 1972. "Vertical Vibrations of Embedded Footings." *Journal of the ASCE Soil Mechanics and Foundations Division* 98 (SM12):1291-1310.

Novak, M. 1974. "Dynamic Stiffness and Damping of Piles." *Canadian Geotechnical Journal* 11:574-598.

Parmelee, R. A., D. S. Perelman, and S. L. Lee. 1969. "Seismic Response of Multistory Structures on Flexible Foundations." *Bulletin of the Seismological Society of America* 29:1061-1070.

Poulos, H. G. and E. H. Davis. 1974. *Elastic Solutions for Soil and Rock Mechanics.* New York: Wiley and Sons.

Rainer, J. H. 1975. "Simplified Analysis of Dynamic Structure-Ground Interaction." *Canadian Journal of Civil Engineering* 2 (3):345-356.

Rainer, J. H. 1975. "Damping in Dynamic Structure-Foundation Interaction." *Canadian Geotechnical Journal* 12:13-22.

Richart, F. E., J. R. Hall, Jr., and R. D. Woods. *Vibrations of Soils and Foundations.* Englewood Cliffs, New Jersey: Prentice-Hall.

Rutenburg, A. 1982. "Simplified *P*-delta Analysis for Asymmetric Structures." *Journal of the Structural Division, ASCE*, 108(ST9).

Seed, H. B., J. Lysmer, and R. Hwang. 1974. "Soil-Structure Interaction Analysis for Seismic Response." *Journal of the ASCE Geotechnical Engineering Division* 101 (GT5):439-457.

Seed, H. B., J. Lysmer, and R. Hwang. 1970. *Soil Moduli and Damping Factors for Dynamic Response Analysis.* Report EERC70-10. Berkeley: n University of California.

Seed, H. B., R. V. Whitman, and J. Lysmer. 1977. "Soil-Structure Interaction Effects in the Design of Nuclear Power Plants." In *Structural and Geotechnical Mechanics, A Volume Honoring N. M. Newmark*, edited by W. J. Hall. Englewood Cliffs, New Jersey: Prentice-Hall.

Skinner, R. I., W. H. Robinson, and G. H. McVerry. 1993. *An Introduction to Seismic Isolation.* Sussex, England: Wiley and Sons.

Skinner, R. I., et al. 1991. "Structures Congress '91," *Proceedings*, Ninth ASCE Structures Congress.

Stokoe, K. H., II, and R. D. Woods. 1972. "In Situ Shear Wave Velocity by Cross-Hole Method." *Journal of the ASCE Soil Mechanics and Foundations Division* 98 (SM5):443-460.

Structural Engineers Association of California. 1968, 1973, 1974, and 1976, and 1987. *Recommended Lateral Force Requirements and Commentary*. San Francisco: SEAOC.

Thomson, W. T. 1965. *Vibration Theory and Application*. New York: Prentice-Hall.

Tsai, N. C. 1974. "Modal Damping for Soil-Structure Interaction." *Journal of the ASCE Engineering Mechanics Division* 100 (EM2):323-341.

Vaish, A. K., and A. K. Chopra. 1974. "Earthquake Finite Element Analysis of Structure-Foundation Systems." *Journal of the ASCE Engineering Mechanics Division* 100 (EM6):1011-1016.

Veletsos, A. S., and V. V. Nair. 1975. "Seismic Interaction of Structures on Hysteretic Foundations." *Journal of the ASCE Structural Division* 101 (ST1):109-129.

Veletsos, A. S. 1977. "Dynamics of Structure-Foundation Systems." In *Structural and Geotechnical Mechanics, A Volume Honoring N. M. Newmark*, edited by W. J. Hall, pp. 333-361. Englewood Cliffs, New Jersey: Prentice-Hall.

Veletsos, A. S., and J. W. Meek. 1974. "Dynamic Behavior of Building Foundation Systems." *Earthquake Engineering and Structural Dynamics* 3 (2):121-138.

Veletsos, A. S., and B. Verbic. 1973. "Vibration of Viscoelastic Foundations." *Earthquake Engineering and Structural Dynamics* 2 (1): 87-102.

Veletsos, A. S., and Y. T. Wei. 1971. "Lateral and Rocking Vibration of Footings." *Journal of the ASCE Soil Mechanics and Foundations Division* 97 (SM9):1227-1248.

Veletsos, A. S., and B. Verbic. 1974. "Basic Response Functions for Elastic Foundations." *Journal of the ASCE Engineering Mechanics Division* 100 (EM2):189-202.

Wiegel, R. L., Ed. 1970. *Earthquake Engineering*. New York: Prentice-Hall.

Wilson, E. L., and A. Habibullah. 1987. "Static and Dynamic Analysis of Multi-story Buildings Including *P*-delta Effects." *Earthquake Spectra* 3(2).

Wilson, E. L., and A. Habibullah. 1987. "Static and Dynamic Analysis of Multi-Story Buildings Including *P*-delta Effects." *Earthquake Spectra* 3(2).

Wilson, E. L., M. I. Suharwardy, and A. Habibullah. 1989. *A Seismic Analysis Method Which Satisfies the 1988 Lateral Force Requirements*. Berkeley, California: Computers and Structures, Inc.

Wilson, E. L., M. Yuan, and J. Dickens. 1982. "Dynamic Analysis by Direct Superposition of Ritz Vectors." *Earthquake Engineering and Structural Dynamics* 10.

Wilson, E. L., Der Kiureghian, and E. P. Bayo. 1981. "A Replacement for the SRSS Method in Seismic Analysis." *Earthquake Engineering and Structural Dynamics* 9.

Appendix to Chapter 2 Commentary

PASSIVE ENERGY DISSIPATION SYSTEMS

PREFACE: The *NEHRP Recommended Provisions* is a resource document, not a model code; therefore, this appendix is included to introduce to potential users new and relevant techniques for incorporating energy dissipation devices into earthquake-resistant building designs and to provide the requirements necessary to ensure that the devices will perform as designed. An independent engineering panel should be used to review each building application of passive energy dissipation systems

The technical basis for Sec. 2A.1 through 2A.3 of this appendix is well established, and energy dissipation devices have been utilized in the seismic rehabilitation of buildings in Canada, Japan, Mexico, and the United States; however, the design requirements provided in this appendix have not been evaluated under typical design office conditions. The technical basis for Sec. 2A.4 has been discussed thoroughly by engineers, researchers, and some current energy dissipation device vendors; however, it is important that no specific device be excluded unnecessarily.

Review of, trial designs based on, and comments on this appendix are encouraged. Please direct such feedback to the BSSC.

2A Provisions for Passive Energy Dissipation Systems: The provisions contained in this Appendix are loosely based on the draft document "Tentative Seismic Design Guidelines for Passive Energy Dissipation Systems" developed by the Energy Dissipation Working Group of the Protective Systems Subcommitee of the Structural Engineers Association of Northern California Seismology Committee.

Acceptable performance of a building during earthquake shaking is predicated on the lateral force resisting system in the building being able to absorb and dissipate energy in a stable manner. For severe earthquake shaking, the energy dissipation system must exhibit stable behavior for a large number of loading cycles. In conventional earthquake-resistant buildings, enery dissipation (damage) occurs in plastic hinge zones in members of the structural frame that routinely form part of the gravity load resisting sytem; such damage to the gravity frame may be irreparable.

The function of an energy dissipation device in an energy dissipation system is to dissipate earthquake-induced energy. The energy dissipation assembly is a one-level, one-bay assembly composed of an energy dissipation device(s) and its (their) lateral and gravity support frame(s). The energy dissipation system is the complete collection of lateral force resisting elements including all energy dissipation devices and all structural elements that transfer force between elements of the energy dissipation system. The relationship between the energy dissipation device and the energy dissipation assembly is shown schematically below in Figure C2A.1.

The energy dissipation devices are typically not a part of the gravity load-resisting system, and can be replaced after an earthquake should it be necessary. The general philosophy of these provisions is similar to that of the provisions for conventional structures and is described in Sec. 2A.1 of this *Commentary*.

The potential advantages of energy dissipation systems and the recent advancements in energy dissipation device products have already lead to the design and construction of buildings and bridges incorporating energy dissipation devices in Canada, Japan, Mexico, New Zealand, and the United States. This activity and concerted research efforts in the United States has provided the opportunity to supplement existing building codes with design guidelines for buildings incorporating energy dissipation devices.

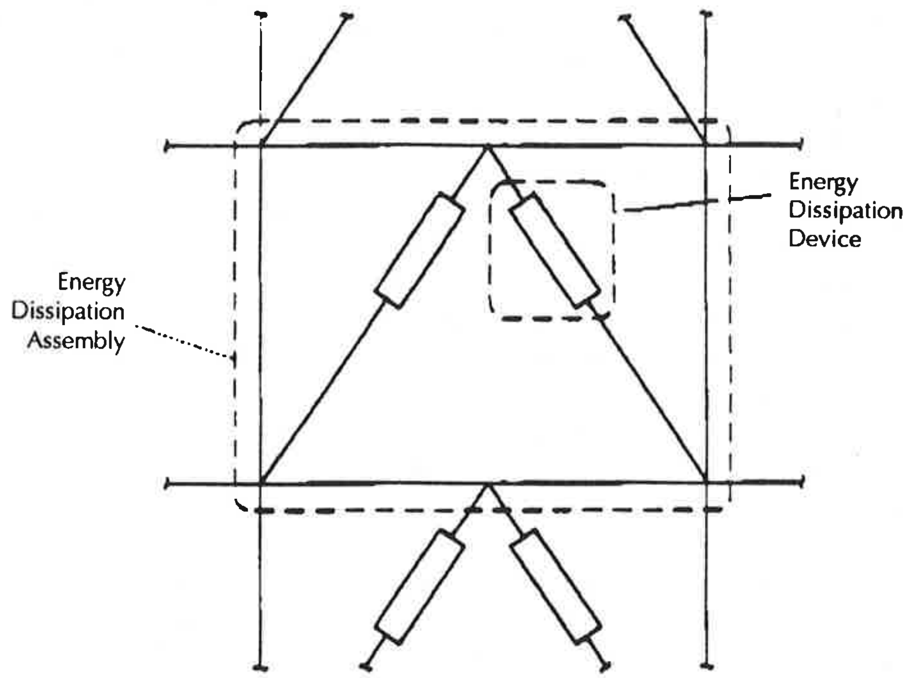


FIGURE C2A.1 Energy dissipation nomenclature.

The *Provisions* provides general design guidelines applicable to a wide range of possible energy dissipation devices rather than addressing specific methods of energy dissipation. By remaining general, the *Provisions* must rely on mandatory testing of system hardware to confirm the engineering parameters used in the design and to verify the adequacy of the energy dissipation system. Some systems may not be capable of demonstrating acceptability by test and consequently should not be permitted.

2A.1 GENERAL: The performance goals that form the basis of these provisions are idential to those of conventional structures, that is, to prevent substantial loss of life by partial or complete building collapse in the design-level earthquake. No special protection against structural or nonstructural damage is sought or implied by the provisions.

The provisions provided in Sec. 2A are minimum requirements for the implementation of energy dissipation systems developed to meet the performance goals noted above. These provisions are to be used in conjunction with Sections 2.2, 2.3, 2.4, 2.5, and 2.6.

Exposure of energy dissipation devices to enviromental effects that include wind-induced member actions, aging effects, high- and low-cycle fatigue, changes in operating temperature, exposure to airborne contaminants and moisture, may lead to either degradation or changes in their mechanical characteristics. Such effects must be accounted for in the design process. For example, consider two types of energy dissipation devices: viscous, and metallic-yielding energy dissipation devices.

a. The force-deformation response of linear viscous or viscoelastic devices may be affected by changes in operating temperature. The designer should bound the response of both the enery dissipation device and the lateral load-resisting system to account for plausible variations in the energy dissipation devices' operating temperature.

b. Metallic yielding devices may be subject to failure by low-cycle fatigue. These types of devices should be designed to remain elastic for the design wind storm and to yield only during moderate to severe earthquake shaking.

2A.2 STRUCTURAL FRAMING SYSTEM: An energy dissipation device is only effective if there is relative displacement and/or relative velocity between the two ends of the device. Given the inventory of lateral load-resisting systems provided in Table 2.2.2, the structural systems best suited for implementation of energy dissipation devices are the moment-resisting space frame and the flexible dual system, in either structural steel or reinforced concrete. The inter-story response of a stiff lateral load-resisting system, such as a reinforced concrete shear wall system or a steel-braced dual system, is generally characterized by both small relative velocities and small relative displacements. As such, it may not be feasible to implement supplemental energy dissipation, or damping devices. However, if it can be demonstrated that sufficient relative motion can be achieved in a lateral load-resisting system other than a moment resisting space frame or a flexible dual system, then the use of energy dissipation devices should be considered.

The primary lateral load-resisting system should be used as the basis for selecting response modification factors (R), deflection amplification factors (C_d), and building height limitations. The primary lateral load-resisting system is the structural system, exclusive of the energy dissipation assemblies, that resist lateral loads. For example, consider a lateral load-resisting system composed of a special steel moment-resisting frame with energy dissipation devices mounted in chevron brace assemblies: the primary lateral load-resisting system is the special steel moment-resisting space frame with a response modification factor of 8, a deflection amplification factor of 5.5, and no limit on building height.

For new construction, the energy dissipation system should be used in parallel with one of the seismic force resisting systems listed in Table 2.2.2.

In order to provide a lateral and torsional redundancy in a building incorporating energy dissipation devices, a minimum of two energy dissipation devices should be provided at each level of the building, in each principal direction of the building. However, if it can be demonstrated by nonlinear time history analysis that satisfactory performance can be achieved with a non-compliant vertical and/or plan distribution of energy dissipation devices, then said distribution of energy dissipation devices should be deemed acceptable. Satisfactory performance is measured herein as compliance with both the drift limits set forth in Table 2.2.7 and member curvature demands less than the member curvature capacities as determined by rational analysis as described below.

2A.3 ANALYSIS PROCEDURES: As a minimum, the structural analysis should be made in accordance with Section 2.2.5 for buildings assigned to either seismic performance categories A or B by Table 1.4.4.

For buildings assigned to seismic performance category C, the procedures of Sec. 2A.3.1 or 2A.3.2.1 should be used in addition to the requirements of Sec. 2.2.5.

For buildings cited in regions of moderate or high seismic risk, that is, buildings assigned to seismic performance category D or E, the dynamic lateral response procedures presented in Sec. 2A.3 shall be used in addition to the requirements of Sec. 2.2.5. The required procedures depend upon the characteristics of the energy dissipation devices; two classifications are used in these provisions:

a. Linear viscous energy dissipation devices and

b. Other energy dissipation devices.

If an energy dissipation system is composed of both linear viscous (Sec. 2A.3.1) and other (Sec. 2A.3.2) energy dissipation devices, the nonlinear analysis procedures of Sec. 2A.3.2 should be followed.

The minimum design base shear force should not be less than the product of the minimum values of Sec. 2.3, 2.4, 2.5, or 2.6, and the damping reduction factors given in Table 2A.3. This requirement is designed to provide a minimum base shear for the design of a building incorporating energy dissipation devices, so as to prevent gross errors by the designer.

The intent of these guidelines is to preclude nonlinear behavior in the structural members in the energy dissipation assembly, exclusive of the energy dissipation devices. As such, structural members in the energy dissipation assembly are to be designed using capacity design procedures for 1.2 times the maximum force in the energy dissipation device(s) at the design-level earthquake. For linear viscous devices, the peak force in the energy dissipation device will be achieved for the maximum velocity in the device. For energy dissipation devices whose mechanical characteristics are frequency-independent, the peak force in the device will be realized at the maximum displacement in the device. For energy dissipation devices that exhibit some frequency dependence, the peak force will be achieved at an intermediate point in the velocity-displacement phase space. If nonlinear analysis is undertaken, the maximum forces in the energy dissipation assemblies are computed explicitly - these forces, multiplied by 1.2, can be used to design the members in the energy dissipation assemblies. If nonlinear analysis is not undertaken, the maximum velocities and displacements in the energy dissipation devices for the design-level earthquake can be estimated using: C_{sm} as computed in Eq. 2.4.5-3; the fundamental frequency of the building (computed on the basis of the effective stiffness of each energy dissipation device at the displacement in the device corresponding to the design-level earthquake); and the displacement amplification factor, C_d, for the primary lateral load-resisting system per Table 2.2.2.

The terms design velocity and design displacement are used throughout the provisions. These two terms correspond to the velocity and displacement in an energy dissipation device at the design-level earthquake, and can be estimated using either nonlinear analysis or by factoring the "design" response quantities (reduced from the elastic level by the factor R) as noted in the last sentence of the preceding paragraph.

2A.3.1 LINEAR VISCOUS DEVICES: The seismic base shear force for buildings designed using linear viscous energy dissipation devices, that is, devices which have a linear force versus relative velocity relationship, may be obtained by multiplying the seismic base shear as determined from Eq. 2.3.2 by the damping reduction factor given in Table 2A.3. The reduction factors are based on research by Wu and Hanson (1989) and are similar to those given by Newmark and Hall (1982).

The tabular values given in Table 2A.3 correctly predict the reductions in the displacement response of a single degree of freedom oscillator afforded by increases in equivalent viscous damping above 5 percent of critical. The same is generally true for modal forces for low and moderate levels of damping. However, the use of the reduction factors listed in Table 2A.3 may underestimate the maximum total modal force for high levels of damping. As such, the reduction factors should be used with care if highly damped systems are being analyzed and designed.

The tabular values for effective modal damping are to be computed as the sum of the damping provided by the supplemental energy dissipation devices, and equivalent viscous damping in excess of 5 percent of critical provided by the structural frame, exclusive of the dampers.

Modal damping beyond 30 percent of critical requires nonlinear time history analysis as provided in Sec. 2A.3.2; this requirement is consistent with provisions for seismic isolation (Sec. 2.6). Note, increases in modal damping beyond 30 percent are unlikely to be cost-effective.

2A.3.2 OTHER ENERGY DISSIPATION DEVICES: For energy dissipation systems incorporating energy dissipation devices other than those covered by Sec. 2A.3.1, the analysis and design of an energy dissipation system is a two-step design process.

The first step treats the system as an equivalent viscously damped system for preliminary design. The second step requires nonlinear time history analysis of the building to verify expected response of the energy dissipation devices, and the forces and deformations in the energy dissipation assemblies and the energy dissipation system.

2A.3.2.1 Equivalent Viscously Damped System: The effective stiffness and effective damping of the energy dissipation system shall be based on the results of prototype device tests performed in accordance with Sec. 2A.4.

For the purposes of analysis and design, the effective stiffness of the energy dissipation device shall be calculated as follows:

$$k^{D_{eff}} = \frac{|F_D^+| + |F_D^-|}{|\Delta^+| + |\Delta^-|} \qquad \text{(C2A.3.2.1a)}$$

where F^+_D and F^-_D are the positive and negative force in the energy dissipation device, recorded at the positive design displacement (Δ^+) and the negative design displacement (Δ^-) in the energy dissipation device, respectively, all as shown in Figure C2.6.9.

The effective stiffnesses of the energy dissipation devices at the corresponding design displacements (for the design-level earthquake) should be used to establish the modal characteristics of the building.

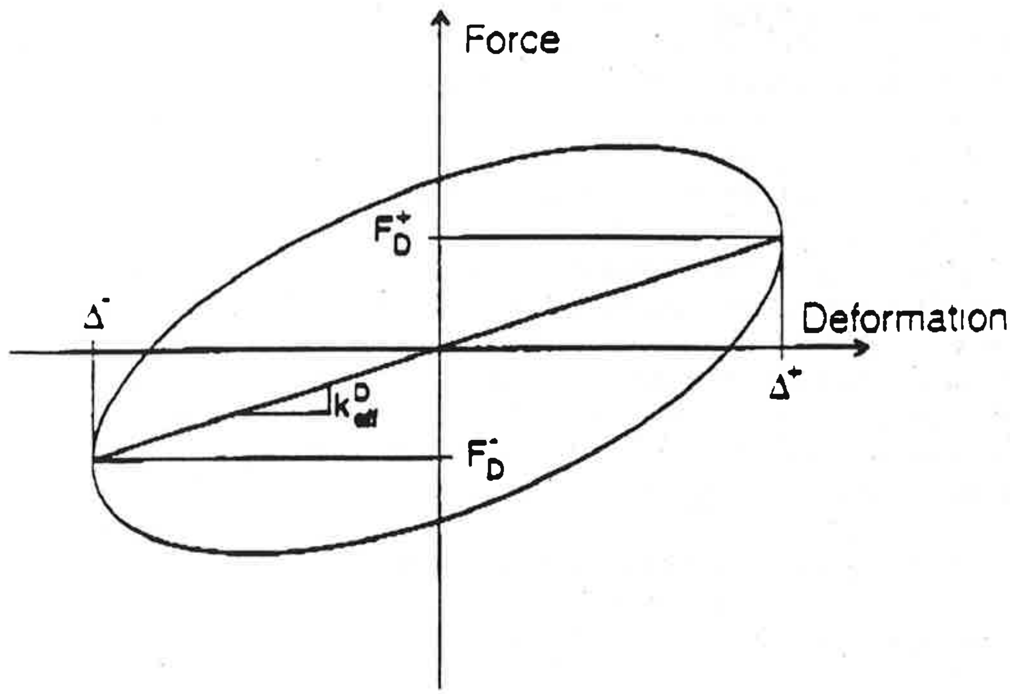


FIGURE C2A.3.2.1 Definition of effective stiffness for an energy dissipation device.

The effective damping coefficient for an energy dissipation device and the energy dissipated per cycle are related as follows:

$$c_{eq} = \frac{W_D T}{2\pi^2 \Delta^2} \qquad \text{(C2A.3.2.1b)}$$

where c_{eq} is the effective damping coefficient, W_D is the energy dissipated per cycle corresponding to a displacement cycle of ($+\Delta$, $-\Delta$) at a loading period of T, T is the loading period, and Δ is the displacement across the device.

In order to calculate the effective modal damping coefficient for an energy dissipation assembly, the displacement, Δ, must be modified to account for the flexibility of the framing supporting the device. Note that modal damping contributions from every energy dissipation assembly at a given level should be aggregated to compute the effective modal damping coefficient at that level.

The equivalent viscous damping in mode i, ξ_i, can be computed using the methodology of Constantinou and Symans (1993). The equivalent viscous damping in any mode can be estimated by:

$$\xi_{str} + \frac{\Sigma W_D}{4\pi SE} \tag{2A.3.2.1c}$$

where ξ_{str} is the equivalent modal damping from the structure, $\sum W_D$ is the sum of the W_D values for all devices at all levels through one cycle of displacement in that mode, and SE is the maximum strain energy in the structure for the same modal displacement. Modal base shears, modal displacements in the energy dissipation assembly, and modal story drifts should be calculated per Sec. 2.4.6 multiplied by the reduction values given in Table 2A.3 for the computed level of model damping.

2A.3.2.2 Nonlinear Dynamic Response Verification: The building design, including the response of the energy dissipation devices, resulting from the application of Sec. 2A.3.2.1 should be verified by nonlinear analysis for the design-level earthquake as given below. The inter-story drifts in the building, as predicted by nonlinear analysis, shall not exceed the drift limits specified in Table 2.2.7.

Structural analysis computer programs routinely compute member actions as the product of the element stiffness and element deformation matrices. Member actions induced by viscous effects will not generally be accounted for. As such, an analysis of a building incorporating viscous or viscoelastic devices may underestimate the maximum actions in some members of the lateral force-resisting system; forces induced by viscous damping must be accounted for in the design process.

The member curvatures, rotations, and drifts computed using nonlinear analysis shall not exceed the capacity of the individual member as calculated using rational design procedures. Sample derivations of moment-curvature relationships for reinforced concrete elements are given by Paulay and Priestley (1992); these principles may be extended to steel structures.

The velocity and displacement response of the individual energy dissipation devices, computed using nonlinear analysis, should form the basis of the prototype testing program described in Sec. 2A.4.1.

2A.3.2.2.1 Ground Motion Records: For two-dimensional nonlinear building analysis, not less than three horizontal ground motion records should be selected by the designer. Each ground motion should be scaled such that each ordinate of its five percent damped response spectrum in the period range T - 0.5 second to T + 0.5 second is not less than the corresponding ordinate in the design spectrum specified in Sec. 2.4, where T includes the periods of all modes of vibration which contribute at least five percent of the total modal mass.

For three-dimensional nonlinear building analysis, not less than three pairs of horizontal ground motion records should be selected and scaled per the corresponding section in the seismic isolation provisions (Sec. 2.6.4.4.2).

The duration of the ground motion time history records should be consistent with the magnitude and source characteristics of the design earthquake. Time histories developed within 15 km (9.3 mi) of an active fault shall include near-fault phenomena consistent with the maximum capable earthquake on the active fault. Table 2.6.3.3.1a in the seismic isolation provisions may be used as a guide for assessing the influence of near-fault effects.

2A.3.2.2.2 Mathematical Model: Three-dimensional mathematical models will generally be required for the design and analysis of buildings incorporating energy dissipation devices. If a building's lateral load resisting system and the energy dissipation system satisfy all the requirements for plan regularity as specified in Sec. 2.2.3.1, two-dimensional mathematical models may be used to analyze and design the building.

The mathematical model should account for both the plan and vertical spatial distribution of the energy dissipation devices throughout the building. If the energy dissipation devices are dependent on loading frequency, temperature, sustained loads, and bi-lateral loads, such dependence should be accounted for in the analysis. In addition, for three-dimensional mathematical models, the energy dissipation system should be modeled in sufficient detail to capture the translational and torsional responses of the building considering the most disadvantageous location of mass eccentricity.

2A.4 TESTING: Access to the energy dissipation devices should be provided for both inspection and possible replacement in the aftermath of an earthquake.

The design guidelines for the implementation of passive energy dissipation devices, presented in Sec. 2A, are applicable to a wide range of device hardware and rely on hardware testing to confirm the engineering parameters used in the design and to verify the adequacy of the energy dissipation system. Energy dissipation devices not capable of demonstrating acceptability by test should be rejected.

2A4.1 PROTOTYPE TESTS: Prototype testing should be undertaken on at least two full-size energy dissipation devices of each predominant type and size used in a building. However, if energy dissipation devices of a similar size and material composition have been previously tested in the manner described in this section, they need not be tested again, provided that the device hardware vendor, or his/her representative, can demonstrate to the engineer-of-record that all of the mechanical characteristics of the energy dissipation device can be similitude-scaled to the specified force and stroke capacity.

If vertical load on the energy dissipation device can influence its force-deformation response, and vertical load will be imposed on the device once installed in the building, due account should be taken of this imposed vertical load during testing.

Each response cycle of the energy dissipation device should be recorded for review by the engineer-of-record. Three levels of testing are mandated for the prototype energy dissipation devices:

a. Two hundred fully-reversed cycles of loading at the level of force in the energy dissipation device computed for the design wind storm;

b. Fifty fully-reversed cycles of loading at the design displacement, that is, the displacement in the device corresponding to the design-level earthquake; and

c. Ten fully-reversed cycles at a displacement corresponding to the maximum capable earthquake, calculated by multiplying the design displacement by the factor M_M given in Table 2.6.3.3.4 in the seismic isolation provisions.

If the force-deformation response of an energy dissipation devise is dependent on loading frequency, the tests noted above also should be performed at loading rates of 0.5, 1.0, and 2.0 times the fundamental frequency of the building, where the fundamental frequency of the building is computed using the effective stiffnesses of the energy dissipation devices computed at their respective design displacements. If the force-deformation response of an energy dissipation device is dependent on bi-lateral deformation, the above-noted tests should be augmented by additional tests performed at a bi-lateral displacement in the energy dissipation device corresponding to the design-level earthquake for the fifty fully-reversed cycles of loading at the design displacement, and the maximum capable earthquake for the ten fully-reversed cycle of loading at M_M times the design displacement.

2A.4.2 FORCE-DEFORMATION CHARACTERISICS: The force-deformation characteristics of an energy dissipation device should be based on the prototype tests described in Sec. 2A.4.1. The energy dissipation afforded by the energy dissipation device should be determined from the area contained within the hysteresis curve. The effective stiffness of the energy dissipation device should be determined using Eq. C2A.3.2.1a.

The force-deformation response of each energy dissipation device should be recorded for all cycles of prototype testing.

Some energy dissipation devices exhibit temperature-dependent response. The analysis and design of an energy dissipation system incorporating temperature-dependent energy dissipation devices should account for this effect by bounding the response of the building through consideration of maximum and minimum effective stiffnesses of the energy dissipation devices, the maximum force transmitted by the energy dissipation device, and the minimum area under the hysteresis curve. The design of a temperature-dependent energy dissipation device should be based on its minimum effective stiffness and minimum area under the hysteresis curve, all as measured during testing.

2A.4.3 SYSTEM ADEQUACY: The performance of a prototype energy dissipation device should be assessed as adequate if the following conditions are satisfied:

a. The force-deformation curves for the tests described in Sec. 2A4.1 have non-negative incremental force-carrying capacities. This requirement applies to frequency-independent energy dissipation devices, such as metallic-yielding and friction-slip devices.

b. For each test sequence specified in Sec. 2A4.1 the effective stiffness of an energy dissipation device for any one cycle should not differ by more than 15 percent from the average effective stiffness as calculated from all tests of that sequence. This requirement is included to preclude the degradation of the stiffness and strength of a freqency-independent energy dissipation device.

   Energy dissipation devices that exhibit frequency-dependent or temperature-dependent hysteretic behavior do not have to comply with this requirement, provided that it is demonstrated by analysis that the variations in effective stiffness measured during testing do not have a deleterious effect on the response of the building. Analysis of a building incorporating such energy dissipation devices should consider possible variations in the stiffness and strength of the energy dissipation devices.

c. For each test sequence of Sec. 2A.4.1 the area of the hysteresis curve for an energy dissipation device for any one cycle does not differ by more than 15 percent from the average area of the hysteresis loop as calculated from all tests of that sequence. This requirement is included to ensure that the assumed level of energy dissipation (damping) is provided for a large number of loading cycles or, conversely, a long duration of earthquake shaking.

   Energy dissipation devices that exhibit frequency-dependent or temperature-dependent hysteretic behavior do not have to comply with this requirement, provided that it is demonstrated by analysis that the variations in the area of the hysteresis loop (equivalent viscous damping) measured during the testing do not have a deleterious effect on the response of the building.

REFERENCES:

Applied Technology Council. 1993. *Proceedings of the Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control* (ATC-17). (Provides an excellent overview of the state-of-the-art in energy dissipation technology.)

Constantinou, M. C., and M. D. Symans. 1993. "Experimental Study of Seismic Response of Buildings with Supplemental Fluid Dampers." In *The Structural Design of Tall Buildings*, Vol. 2, pp. 93-132.

Newmark, N. M., and W. J. Hall. 1982. Monograph in *Earthquake Spectra and Design*, Earthquake Engineering Research Institute.

Paulay, T., and M. J. N. Priestley. 1992. *Seismic Design of Reinforced Concrete and Masonry Buildings*. New York: John Wiley & Sons, Inc.

Wu, J. P., and R. D. Hanson. 1989. "Inelastic Response Spectra with High Damping," ASCE *Journal of the Structural Division*, 115(6):1412-1431.415

Chapter 3 Commentary

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENTS DESIGN REQUIREMENTS

3.1 GENERAL: The general requirements establish minimum design levels for architectural, mechanical, electrical, and other nonstructural systems and components (hereinafter referred to as "components") recognizing occupancy use, occupant load, need for operational continuity, and the interrelation of structural and architectural, mechanical, electrical, and other nonstructural components. Several exemptions are made to the *Provisions*:

1. All components in Seismic Performance Category A are exempted because of the lower seismic input for these items

2. All mechanical and electrical components in Seismic Performance Categories B and C are exempted if they have an importance factor (I_p) equal to 1.00 because of the low acceleration and the classification that they do not contain hazardous substances and are not required to function to maintain life-safety.

3. All components in all Seismic Performance Categories, weighing less than 400 pounds (1780 N), and are mounted 4 ft (1.22 m) or less above the floor are exempted if they have an importance factor (I_p) equal to 1.00, because they do not contain hazardous substances, are not required to function to maintain life safety, and are not considered to be mounted high enough to be a life-safety hazard if they fell.

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. 3.1.3. Vertical forces on mechanical and electrical components are specified in Sec. 3.3.2.

In the design and evaluation of support structures and the attachment of the architectural component, flexibility should be considered. Components that are subjected to seismic relative displacements (i.e., components that are connected to both the floor and ceiling level above) should be designed with adequate flexibility to accommodate imposed displacements. In the design and evaluation of equipment support structures and attachments, flexibility will reduce the fundamental frequency of the supported equipment and increase the amplitude of its induced relative motion. This lowering of the fundamental frequency of the supported component often will bring it into the range of the fundamental frequency of the supporting building or into the high energy range of the input motion. In evaluating the flexibility/stiffness of the component attachment, the load path in the components should be considered especially in the region near the anchor points.

Although the components included in Tables 3.2.2 and 3.3.2 are listed separately, significant interrelationships exist among 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 ultimately may 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 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 3.2.2 or 3.3.2 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 components that are in contact with or in close proximity to structural or other nonstructural components must be given special study to avoid damage or failure when seismic motion occurs. An example is where an important element, 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 components 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 component.

3.1.2 COMPONENT FORCE TRANSFER FACTOR: 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 structure.

If an architectural component were to fail during an earthquake, the mode of failure probably would be related to faulty design of the component, interrelationship with another component that fails, interaction with the structural framing, 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 infill walls--may interact with the structural framing and be forced to act structurally as a result of excessive building deformation. The build up 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 anchored directly 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) often is not restrained at all, and electrical equipment other than generators, which are normally isolated to dampen vibrations, usually is 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 produced solely by the effects of gravity cannot be counted on to resist seismic forces as equipment and fixtures often tend to "walk" due to rocking when subjected to earthquake motions. This often is accentuated by the vertical ground motions. Because frictional resistance cannot be relied upon, positive restraint must be provided for each component.

3.1.3 SEISMIC FORCES: The design seismic force is dependent upon the weight of the system or component, the component amplification factor, the component acceleration at point of attachment to the structure, the component importance factor, and the component response modification factor.

The seismic design force equations presented originated with a study and workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) with funding from the National Science Foundation (NSF) (Bachman et al., 1993) The participants examined recorded acceleration data in response to strong earthquake motions. The objective was to develop a "supportable" design force equation that considered actual earthquake data as well as component location in structure, component anchorage ductility, component importance, component safety hazard upon separation from the structure, structural response, site conditions, and seismic zone. The participants and BSSC Technical Subcommittee 8 believe that Eq. 3.1.3-2 achieves the objectives without unduly burdening the practitioner with complicated formulations.

The component amplification factor (a_p) represents the dynamic amplification of the component relative to the fundamental period of the structure (T). It is recognized that at the time the components are designed or selected, the structural fundamental period is not always defined or readily available. It is also recognized that the component fundamental period (T_p) is usually only accurately obtained by expensive shake-table or pull-back tests. A listing is provided of a_p values based on the expectation that the component will usually behave in either a rigid or flexible manner. In general, if the fundamental period of the component is less than 0.06 sec, no dynamic amplification is expected. It is not the intention of the Provisions to preclude more accurate determination of the component amplification factor when reasonably accurate values of <u>both</u> the structural and component fundamental periods are available. Figure C3.1.3-1 is from the NCEER work and is an acceptable formulation for a_p as a function of T_p/T.

The component acceleration coefficient at the point of attachment to the structure (A_p) represents the base acceleration of the structure amplified by an expression appropriate to the dynamic characteristics of the structure. Examination of recorded structure acceleration data in response to large earthquakes reveals that structures typically exhibit an increase in floor acceleration response near the top of the structure, especially for flexible structures. The data also reveal that for most structures, the floor acceleration response is fairly constant from the second floor to near the roof. Eq. 3.1.3-3 and 3.1.3-4 are intended to assign design accelerations at the point of attachment of the component that are consistent with recorded data and anticipated earthquake response for the building design earthquake ground motion.

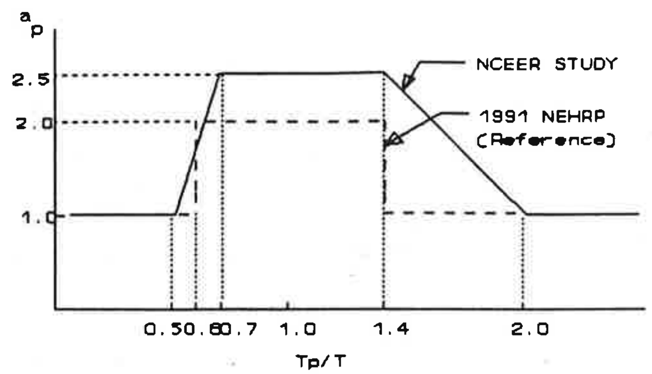


FIGURE C3.1.3-1 NCEER formulation for a_p as function of structural and component periods.

The component response modification factor (R_p) represents the energy absorption capability of the component's structure and attachments. As with the structure modification factor (R), these values are judgmentally determined utilizing the collective wisdom and experience of the responsible committee. In general, the following benchmark values were used:

R_p = 1.5, brittle or buckling failure mode expected

R_p = 3.0, some minimal level of energy absorption capability

R_p = 4.0, ductile materials and detailing

R_p = 6.0, highly ductile materials and detailing

Eq. 3.1.3-2 represents a trapezoidal distribution of floor accelerations within the structure, linearly varying from the acceleration at the ground (A_g) to the acceleration at the roof (A_r). The ground acceleration (A_g) is intended to be the same acceleration used as design input for the structure itself and will include site effects. Figure C3.1.3-2 indicates the recorded data average and the recorded data average plus one standard deviation. Eq. 3.1.3-3 approximates the recorded data average plus one standard deviation when the roof acceleration equals the maximum of four times the ground acceleration. Therefore, the maximum design equation will bound 84 percent of the recorded data. This was judged to be reasonable for design.

Eq. 3.1.3-3 represents the maximum acceleration felt by a component on the structure roof. Past practice has accepted that a reasonable value for A_r is twice the structure acceleration (A_s). This value is limited to $4.0A_g$ to reasonably bound data analyzed in the NCEER study. Figure C3.1.3-3 indicates the comparison of Eq. 3.1.3-3 for different structural periods.

A lower limit for F_p is set to assure a minimal seismic design force. Eq. 3.1.3-5 is the minimum value for F_p determined by setting the quantity a_pA_p/R_p equal to $0.5A_g$.

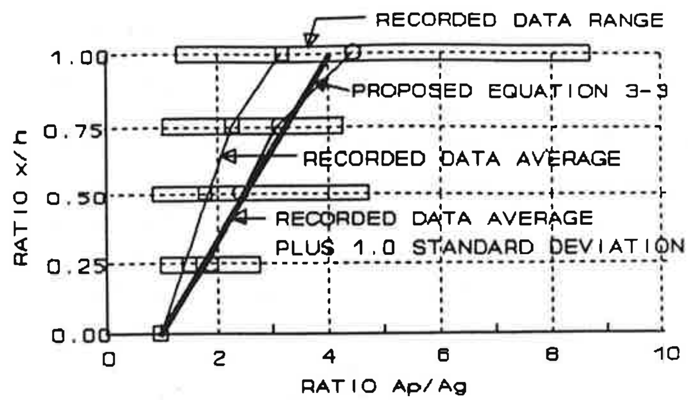


FIGURE C3.1.3-2 Comparison of Eq. 3.1.3-3 with recorded data.

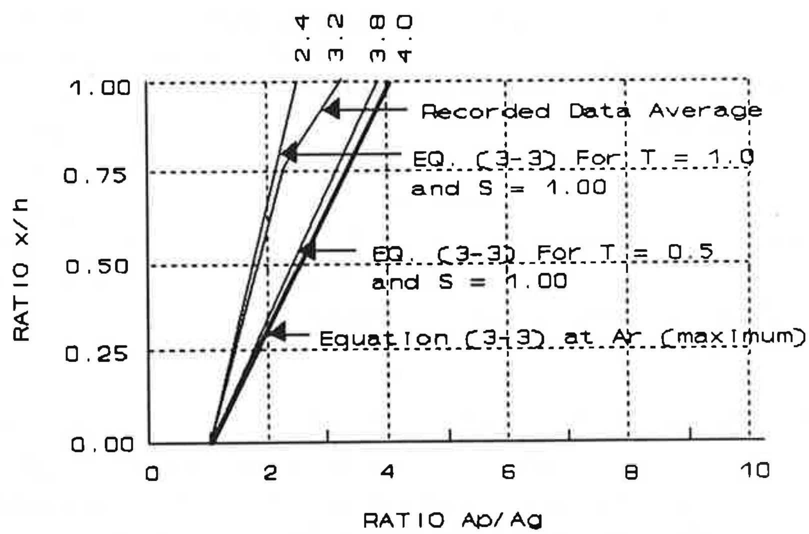


FIGURE C3.1.3-3 Comparison of Eq. 3.1.3-3 at different values for structural period.

To meet the need for a simpler formulation, a conservative maximum value for F_p also was set. Eq. 3.1.3-1 is the maximum value for F_p determined by setting the quantity a_pA_p/R_p equal to 4.0. Eq. 3.1.3-1 also serves as a reasonable "cutoff" equation to assure that the multiplication of the individual factors does not yield an unreasonably high design force.

3.1.4 SEISMIC RELATIVE DISPLACEMENTS: The seismic relative displacement equations were developed as part of the NCEER/NSF study and workshop described above. It was recognized that displacement equations were needed to support the design of cladding, stairwells, windows, piping systems, sprinkler components, and other components that are connected to the structure(s) at multiple levels or points of connection.

Two equations are given for each situation. Eq. 3.1.4-1 and Eq. 3.1.4-3 yield "real" structural displacements as determined by elastic analysis, with no structural response modification factor (R) included. Recognizing that elastic displacements are not always defined or available at the time the component is designed or procured, default Eq. 3.1.4-2 and Eq. 3.1.4-4 also are provided that allow the use of structure drift limitations. Use of these default equations must balance the need for a timely component design/procurement with the possible conservatism of their use. It is the intention that the lesser of the paired equations be acceptable for use.

The designer also should consider other situations where seismic relative displacements could impose unacceptable stresses on a component or system. One such example would be a component connecting two pieces of equipment mounted in the same building at the same elevation, where each piece of equipment has it's own displacements relative to the mounting location. In this case, the designer must accommodate the total of the separate seismic displacements relative to the equipment mounting location.

For some items such as ductile piping, relative seismic displacements between support points generally are of more significance than forces. Piping made of ductile materials such as steel or copper can accommodate relative displacements by local yielding but with strain accumulations well below failure levels. However, components made of less ductile materials can only accommodate relative displacement effects by use of flexible connections or avoiding local yielding. It is further the intent of the *Provisions* to consider the effects of seismic support relative displacements and displacements caused by seismic force on mechanical and electrical component assemblies such as piping systems, cable and conduit systems, and other linear systems, most typically, and the equipment they attach to. Impact of components should also be avoided although ductile materials have been shown to be capable of accommodating fairly significant impact loads. With protective coverings, ductile mechanical and electrical components and many more fragile components can be expected to survive all but the most severe impact loads.

3.1.5 COMPONENT IMPORTANCE FACTOR: The component importance factor (I_p) represents the greater of the life-safety importance of the component and the hazard exposure importance of the structure. This factor indirectly accounts for the functionality of the component or structure by requiring design for a higher force level. It generally is assumed that a better, more functional component will be provided. This may not be sufficient for all components. Transfer of technology from approaches presently in use by the Department of Energy (DOE) and the Nuclear Regulatory Commission (NRC) should be considered by the engineer and/or the owner when unacceptable consequences of failure are anticipated.

3.2 ARCHITECTURAL COMPONENT DESIGN:

3.2.1 GENERAL: The primary focus of the *Provisions* is on the design of attachments, connections, and supports for architectural components.

"Attachments" are means by which components are secured or restrained to the seismic force resisting system of the structure. Such attachments and restraints may include anchor bolting, welded connections, and fasteners.

"Architectural component supports" are those members or assemblies of members, including braces, frames, struts and attachments, that transmit all loads and forces between the component and the building structure. Architectural component supports also transmit lateral forces and/or provide structural stability for the component to which they connect.

The *Provisions* are intended to reduce the threat of life safety hazards posed by components and elements from the standpoint of stability and integrity. There are several circumstances where such components may pose a threat.

1. Where loss of integrity and/or connection failure under seismic motion poses a direct hazard in that the components may fall on building occupants.

2. Where loss of integrity and/or connection failure may result in a hazard for people outside of a building in which components such as exterior cladding and glazing may fall on them.

3. Where failure or upset of interior components may impede access to a required exit.

The *Provisions* are intended to apply to all of the circumstances listed above. Although the safety hazard posed by exterior cladding is obvious, judgment may be needed in assessing the extent to which the requirements should be applied to other hazards.

Property loss through damage to architectural components is not specifically addressed in the *Provisions*. Function and operation of a building also may be affected by damage to architectural components if it is necessary to cease operations while repairs are undertaken. In general, provisions to improve life-safety also will reduce property loss and loss of building function.

In general, functional loss is more likely to be affected by loss of mechanical or electrical components. Architectural damage, unless very severe, usually can be accommodated on a temporary basis. Very severe architectural damage results from excessive structural response that often also results in significant structural damage and building evacuation.

3.2.2 ARCHITECTURAL COMPONENT FORCES: The restriction on R_p values in Footnote *b* to Table 3.2.2 is because of the concern for nonductile failure modes in the component anchorage. Anchorages that could be reasonably expected to fail in a nonductile manner should be designed using $R_p = 1.5$. Chemical anchors and cast-in-place anchor bolts with an embedment length-to-diameter ratio of 8 or less should be considered to be "shallow" anchors.

3.2.3 ARCHITECTURAL COMPONENT DEFORMATION: Specific requirements for cladding are provided. Glazing, both exterior and interior, and partitions must be capable of accommodating story drift without causing a life-safety hazard. Design judgment must be used with respect to the assessment of life-safety hazard and the likelihood of life-threatening damage. Special detailing to accommodate drift for typical replaceable gypsum board or demountable partitions is not likely to be cost-effective, and damage to these components has a low life-safety hazard. Nonstructural fire-resistant enclosures and fire-rated partitions may require some special detailing to ensure that they retain their integrity. Special detailing should provide isolation from the adjacent or enclosing structure for deformation equivalent to the calculated drift (relative displacement). In-plane differential movement between structure and wall is permitted. Provision also must be made for out-of-plane restraint. These provisions are particularly important in relation to the larger drifts experienced in steel or concrete moment frame structures. The problem is less likely to be encountered in stiff shear wall structures.

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 design of exterior walls.

3.2.4 EXTERIOR WALL PANEL CONNECTIONS: The *Provisions* document requires that precast or prefabricated nonbearing wall panels that are attached to or enclose the structure shall be designed to resist the (inertial) forces and shall accommodate movements of the structure resulting from lateral forces or temperature change. The force requirements often overshadow the importance of allowing for moisture or thermal movement.

Connections should be designed such that, if they were to yield, they would do so in a ductile manner without loss of load-carrying capacity. Between points of connection, panels should be separated from the building structure to avoid contact under seismic action.

The *Provisions* document requires allowance for story drift. This required allowance can be 2 in. (51 mm) or more from one floor to the next and may present a greater challenge to the designer than provisions for the forces. In practice, separations between panels are usually limited to about 3/4 in. (19 mm), with the intent of limiting contact under all but extreme building response, and providing for practical joint detailing with acceptable appearance.

If wind loads govern, connectors and panels should allow for not less than two times the story drift caused by wind loads determined using a return period appropriate to the site location.

The *Provisions* requirements are in anticipation of frame yielding to absorb energy. The isolation can be achieved by using slots, but the use of long rods that flex is preferable because this approach is not dependent on installation precision to achieve the desired action. The rods must be designed to carry tension and compression in addition to induced flexural stresses. For floor-to-floor wall panels, the panel usually is rigidly fixed to and moves with the floor structure nearest the panel bottom. In this condition, the upper attachments become isolation connections to prevent building movement forces from being transmitted to the panels. and thus the panel translates with the load supporting structure. The panel also can be supported at the top with the isolation connection at the bottom.

When determining the length of slot or displacement demand for the connection, the cumulative effect of tolerances in the supporting frame and cladding panel must be considered.

The *Provisions* requires that fasteners be designed for 4 times the required panel force and that the connecting member be ductile. This is intended to ensure that the energy absorption takes place in the connecting member and not at the connection itself and that the more brittle fasteners remain essentially elastic under seismic loading. The factor 4 incorporates consideration of installation and material variability.

To minimize the effects of thermal movements and shrinkage on architectural cladding panels, the connection system generally is statically determinant. As a result, cladding panel support systems often lack redundancy and failure of a single connection can have catastrophic consequences.

3.2.5 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, nonstructural walls, and partitions. 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 in Sec. 3.2.2.

Judgment must be used in assessing the deflection capability of the component. The intent is that a heavy material (such as concrete block) or an applied finish (such as brittle heavy stone or tile) should not fail in a hazardous manner as a result of out-of-plane forces. Deflection in itself is not a hazard. A steel-stud partition might suffer considerable deflection without creating a hazard; but if the same partition supports a marble facing, a hazard might exist and special detailing may be necessary.

3.2.6 SUSPENDED CEILINGS: Suspended ceiling systems usually are fabricated using a wide range of building materials with individual components having different material characteristics. Some systems are homogeneous whereas others incorporate suspension systems with acoustic tile or lay-in panels. Seismic performance during recent large California earthquakes has raised two concerns:

a. The support of the individual panels at walls and expansion joints and

b. The interaction with fire sprinkler systems.

The four alternate methods provided have been developed in a cooperative effort by architects, structural engineers, the ceiling industry, and the fire sprinkler industry in an attempt to address these concerns. The four alternates are an improvement over the 1991 *Provisions*. It is hoped that further research and investigation will result in further improvements in future editions of the *Provisions*.

There are four methods in the Provisions to comply with the design and construction requirements of this section:

Method 1: Comply with CISCA (Sec. 3.2.6.3)
Method 2: Displacement Control (Sec. 3.2.6.4)
Method 3: Braced Construction (Sec. 3.2.6.5)
Method 4: Integral Construction (Sec. 3.2.6.6)

Only one method need be complied with.

Consideration shall be given to the placement of seismic bracing and the relation of light fixtures and other loads placed into the ceiling diaphragm and the independent bracing of partitions in order to effectively maintain the performance characteristics of the ceiling system. The ceiling system may require bracing and allowance for the interrelationship of components.

Sec. 3.2.6.3 has been rewritten to include the requirements of the Ceiling and Interior Systems Construction Association (CISCA) and the ASTM Standards for Manufacturing and Installing Metal Ceiling Suspension Systems and Acoustical Tile Lay-In Panels provided that vertical compression struts also are installed.

Dynamic testing of suspended ceiling systems constructed according to the requirements of *UBC* Standard No. 47-18 has demonstrated that the splayed wire even with the vertical compression strut may not adequately limit lateral motion of the ceiling system due to the flexibility introduced by the straightening of the wire end loops. In addition, splay wires usually are installed slack to prevent unleveling of the ceiling grid and to avoid above-ceiling utilities. Not infrequently, bracing wires are omitted because of obstructions. Testing also has shown that system performance without splayed wires or struts was good if adequate width of closure angles and penetration clearance was provided.

The lateral seismic restraint for a nonrigidly braced suspended ceiling is primarily provided by the ceiling coming in contact with the perimeter wall. The wall provides a large contact surface to restrain the ceiling. The key to good seismic performance is that the width of the closure angle around the perimeter is adequate to accommodate ceiling motion and that penetrations, such as columns and piping, have adequate clearance to avoid concentrating restraining loads on the ceiling system. The behavior of an unbraced ceiling system is similar to that of a pendulum; therefore, the lateral displacement is approximately proportional to the level of velocity-controlled ground motion and the square root of the suspension length. Therefore, a new section has been added that permits exemption from force calculations if certain displacement criteria are met. The default displacement limit has been determined based on anticipated damping and energy absorption of the suspended ceiling system assuming minimal significant impact with the perimeter wall.

The derivation of the limits in Sec. 3.2.6.4.2 is as follows: It is assumed that an unbraced suspended ceiling will behave basically like a suspended pendulum. The natural period for a pendulum is given by the equation:

$$T = 2\pi\sqrt{\frac{h}{g}} \qquad \text{(C3.2.6-1)}$$

where h = height of the pendulum and g = acceleration of gravity.

The relative displacement of a single-degree-of-freedom system (such as a pendulum) is given by:

$$S_d = g\left(\frac{T}{2\pi}\right)^2 S_a \qquad \text{(C3.2.6-2)}$$

where S_a = spectral acceleration at period T in g's.

The spectral acceleration as a function of period in the constant velocity range of the spectrum is approximately indicated by the following equation:

$$S_a = \frac{C_v}{T} \qquad \text{(C3.2.6-3)}$$

where

C_v = the spectral acceleration at a period of 1 sec including local site effects.

Substituting, we obtain the following equations:

$$S_d = \frac{hgS_a}{g} = \frac{hC_v}{T} = \frac{C_v\sqrt{gh}}{2\pi} \qquad \text{(C3.2.6-4)}$$

It is assumed that by accounting for increased damping and limited ductility effects, the suspended ceiling design displacements can be given by:

$$\Delta = \frac{C_v \sqrt{gh}}{20} \tag{C3.2.6-5}$$

The width of the closure is taken as twice the displacement to account for displacement in both directions and amplification effects. The width of "rattlespace" gap is taken as half the displacement to account for limited ductility and flexibility of the protrusions such as sprinkler heads, etc.

3.2.7 ACCESS FLOORS: Performance of computer access floors during past earthquakes and during cyclic load tests indicate that typical raised access floor systems may behave in a brittle manner and exhibit little reserve capacity beyond initial yielding or failure of critical connections. Recent testing indicates that individual panels may "pop out" of the supporting grid during seismic motions. Consideration should be given to mechanically fastening the individual panels to the supporting pedestals or stringers in egress pathways.

It is acceptable practice for systems with floor stringers to calculate the seismic force F_p for the entire access floor system within a partitioned space and then distribute the total force to the individual braces or pedestals. Stringerless systems need to be evaluated very carefully to ensure a viable seismic load path.

Overturning effects for the design of individual pedestals is a concern. Each pedestal usually is specified to carry an ultimate design vertical load greatly in excess of the W_p used in determining the seismic force F_p. It is nonconservative to use the design vertical load simultaneously with the design seismic force when considering anchor bolts, pedestal bending, and pedestal welds to base plate. The maximum concurrent vertical load when considering overturning effects is therefore limited to the W_p used in determining F_p. "Slip on" heads are not mechanically fastened to the pedestal shaft and provide doubtful capacity to transfer overturning moments from the floor panels or stringers to the pedestal.

To preclude brittle failure behavior, each element in the seismic load path must demonstrate the capacity for elastic or inelastic energy absorption. Buckling failure modes also must be prevented. Lesser seismic force requirements are deemed appropriate for access floors designed to preclude brittle and buckling failure modes.

3.2.8 CEILING HEIGHT PARTITIONS: Partitions are sometimes designed to run only from floor to a suspended ceiling which provides doubtful lateral support. Partitions subject to these provisions must have independent lateral support bracing from the top of the partition to the building structure or to a substructure attached to the building structure.

3.2.9 STEEL STORAGE RACKS: The Rack Manufacturers Institute has developed and maintained a specification that is utilized by much of the storage rack industry. An attempt has been made to incorporate this specification in the 1994 *Provisions* in a manner that ensures the applicable requirements of the *Provisions* also are met. All storage racks can be designed in accordance with the Rack Manufacturers Institute specification provided that design force requirements are not less than those required by the force requirements for architectural systems and components.

In addition, storage racks located at grade elevation may be designed to the same requirements as building structures provided that **all** the force and detailing requirements of Chapters 2 and 5 are met. R may be taken as 6 unless a higher value can be justified by independent test results.

Based on storage rack performance experienced during the 1994 Northridge earthquake, it is judged to be necessary to account for 67 percent of the rated rack load in determining the seismic weight.

3.3 MECHANICAL AND ELECTRICAL COMPONENT DESIGN:

3.3.1 GENERAL: The primary focus of these provisions is on the design of attachments and equipment supports for mechanical and electrical components.

The provisions are intended to reduce the hazard to life posed by the loss of component structural stability or integrity. The provisions should increase the reliability of component operation but do not directly address the assurance of functionality.

The design of mechanical and electrical components must consider two levels of earthquake safety. For the first safety level, failure of the mechanical or electrical component itself poses no significant hazard. In this case, the only hazard posed by the component is if the support and the means by which the component and its supports are attached to the building or the ground fails and the component could slide, topple, fall, or otherwise move in a manner that creates a hazard for persons nearby. In the first category, the intent of these provisions is only to design the support and the means by which the component is attached to the structure, defined in the Glossary as "equipment supports" and "attachments." For the second safety level, failure of the mechanical or electrical equipment itself poses a significant hazard. In this case, failure could either be to a containment having hazardous contents or contents required after the earthquake or failure could be functional to a component required to remain operable after an earthquake. In this second category, the intent of these provisions is to provide guidance for the design of the component as well as the means by which the component is supported and attached to the structure. The provisions should increase the survivability of this second category of component but the assurance of functionality may require additional considerations.

Examples of this second category include fire protection piping or an uninterruptible power supply in a hospital. Another example involves the rupture of a vessel or piping that contains sufficient quantities of highly toxic or explosive substances such that a release would be hazardous to the safety of building occupants or the general public. In assessing whether failure of the mechanical or electrical equipment itself poses a hazard, certain judgments may be necessary. For example, small flat-bottom tanks themselves may not need to be designed for earthquake loads; however, numerous seismic failures of large flat-bottom tanks and the hazard of a large fluid spill suggest that many, if not most, of these should be. Distinguishing between large and small, in this case, may require an assessment of potential damage caused by a spill of the fluid contents over and above the guidance offered in Sec. 3.3.9.

It is intended that the provisions provide guidance for the design of components for both conditions in the second category. This is primarily accomplished by increasing the design forces with an importance factor, I_p. However, this only affects structural integrity and stability directly. Function and operability of mechanical and electrical components may only indirectly be affected by increasing design forces. For complex components, testing or experience may be the only reasonable way to improve the assurance of function and operability. On the basis of past earthquake experience, it may be concluded that if structural integrity and stability are maintained, function and operability after an earthquake will be reasonably provided for most types of equipment components. On the other hand, mechanical joints in containment components (tanks, vessels, piping, etc.) may not remain leaktight in an earthquake even if after the earthquake leaktightness is re-established. Judgment may suggest a more conservative design related in some manner to the perceived hazard than would otherwise be provided by these provisions.

It is not intended that all equipment or parts of equipment be designed for seismic forces. Determination of whether these provisions need to be applied to the design of a specific piece of equipment or a part of that equipment will sometimes be a difficult task. Damage to or even failure of a piece or part of a component is not a concern of these provisions so long as a hazard to life does not exist. Therefore, the restraint or containment of a falling, breaking, or toppling component or its parts by the use of bumpers, braces, guys, wedges, shims, tethers, or gapped restraints often may be an acceptable approach to satisfying these provisions even though the component itself may suffer damage. Judgment will be required if the intent of these provisions is to be fulfilled. The following example may be helpful: Since the threat to life is a key consideration, it should be clear that a nonessential air handler package unit that is less than 4 ft (1.2 m) tall bolted to a mechanical room floor is not a threat to life as long as it is prevented from significant motions by having adequate anchorage. Therefore, earthquake design of the air handler itself need not be performed. However, most engineers would agree that a 10-ft (3.0 m) tall tank on 6-ft (1.8 m) angles used as legs mounted on the roof near a building exit does pose a hazard. It is the intent of these provisions that the tank legs, the connections between the roof and the legs, the connections between the legs and the tank, and possibly even the tank itself be designed to resist earthquake forces. Alternatively, restraint of the tank by guys or bracing could be acceptable.

It is not the intent of these provisions to require the seismic design of shafts, buckets, cranks, pistons, plungers, impellers, rotors, stators, bearings, switches, gears, nonpressure retaining casings and castings, or similar items. When the potential for a hazard to life exists, it is expected that design efforts will focus on equipment supports including base plates, anchorages, support lugs, legs, feet, saddles, skirts, hangers, braces, or ties.

Many mechanical and electrical components consist of complex assemblies of mechanical and/or electrical parts that typically are manufactured in an industrial process that produces similar or identical items. Such equipment may include manufacturer's catalog items and often are designed by empirical (trial-and-error) means for functional and transportation loadings. A characteristic of such equipment is that it may be inherently rugged. Rugged, as used herein, refers to an ampleness of construction that renders such equipment the ability to survive strong motions without significant loss of function. By examining such equipment, an experienced design professional usually should be able to confirm such ruggedness. The results of equipment ruggedness assessment then will determine the need for an appropriate method and extent of the seismic design or qualification efforts.

It also is recognized that a number of professional and industrial organizations have developed nationally recognized codes and standards for the design and construction of specific mechanical and electrical components. In addition to providing design guidance for normal and upset operating conditions and various environmental conditions, some have developed earthquake design guidance in the context of the overall mechanical or electrical design. It is the intent of these provisions that such codes and standards having earthquake design guidance be used as it is to be expected that the developers have a greater familiarity with the expected failure modes of the components for which their design and construction rules are developed. In addition, even if such codes and standards do not have earthquake design guidance, it is generally regarded that construction of mechanical and electrical equipment to nationally recognized codes and standards such as those approved by the American National Standards Institute provide adequate strength (with a safety margin often greater than that provided by structural codes) to accommodate all normal and upset operating loads. In this case, it could also be assumed that the component has sufficient strength (especially if constructed of ductile materials) to not break up or break away from its supports in such a way as to provide a life-safety hazard. Earthquake damage surveys confirm this.

Specific guidance for selected components or conditions is provided in Sec. 3.3.6 through 3.3.16.

3.3.2 MECHANICAL AND ELECTRICAL COMPONENT FORCES AND DISPLACEMENTS: The restriction on R_p values in the footnote to Table 3.3.2 is because of the concern for nonductile failure modes in the component anchorage. Anchorages that could be reasonably expected to fail in a nonductile manner should be designed using R_p = 1.5. Chemical anchors and cast-in-place anchor bolts with an embedment length-to-diameter ratio of 8 or less should be considered to be "shallow" anchors.

3.3.3 MECHANICAL AND ELECTRICAL COMPONENT PERIOD: Determination of the fundamental period of an item of mechanical or electrical equipment using analytical or in-situ testing methods can become very involved and can produce nonconservative results (i.e., underestimated fundamental periods) if not properly performed.

When using analytical methods, it is absolutely essential to define in detail the flexibility of the elements of the equipment base, load path, and attachment to determine K_p. This base flexibility typically dominates equipment component flexibility and thus fundamental period.

When using test methods, it is necessary to ensure that the dominant mode of vibration of concern for seismic evaluation is excited and captured by the testing. This dominant mode of vibration typically cannot be discovered in equipment in-situ tests that measure only ambient vibrations. In order for the highest fundamental period dominant mode of vibration to be excited by in-situ tests, relatively significant input levels of motion are required (i.e., the flexibility of the base and attachment needs to be exercised).

Many types of mechanical equipment components have fundamental periods below 0.06 sec and may be considered to be rigid. Examples include horizontal pumps, engine generators, motor generators, air compressors, and motor driven centrifugal blowers. Other types of mechanical equipment also are very stiff but may have fundamental periods up to approximately 0.125 sec. Examples of these mechanical equipment items include vertical immersion and deep well pumps, belt driven and vane axial fans, heaters, air handlers, chillers, boilers, heat exchangers, filters, and evaporators. These fundamental period estimates do not apply when the equipment is on vibration-isolator supports.

Electrical equipment cabinets can have fundamental periods of approximately 0.06 to 0.3 sec depending upon weight, stiffness of the enclosure assembly, flexibility of the enclosure base, and load path through to the attachment points. Tall and narrow motor control centers and switchboards lie in the upper end of this period range. Low and medium-voltage switchgear, transformers, battery chargers, inverters, instrumentation cabinets, and instrumentation racks usually have fundamental periods ranging from 0.1 to 0.2 sec. Braced battery racks, stiffened vertical control panels, benchboards, electrical cabinets with top bracing, and wall-mounted panelboards have fundamental periods ranging from 0.06 to 0.1 sec.

3.3.4 MECHANICAL AND ELECTRICAL COMPONENT ATTACHMENTS: For some items such as piping, relative seismic displacements between support points generally are of more significance than inertial forces. Components made of ductile materials such as steel or copper can accommodate relative displacement effects by inelastically conforming to the supports' conditions. However, components made of less ductile materials can only accommodate relative displacement effects by providing flexibility or flexible connections.

Of most concern are distribution systems that are a significant life-safety hazard and are routed between two separate building structures. Ductile components with bends and elbows at the building separation point or components that will be subject to bending stresses rather than direct tensile loads due to differential support motion, are not so prone to damage and are not so likely to fracture and fall. This is valid if the supports can accommodate the imposed loads.

3.3.5 COMPONENT SUPPORTS: It is the intent of these provisions to assure that all mechanical and electrical component supports, the means by which a component transfers seismic loads to the structure, be designed to accommodate the force and displacement effects prescribed. Component supports are differentiated here from component attachments to emphasize that the supports themselves, the structural members, braces, frames, skirts, legs, saddles, pedestals, cables, guys, stays, snubbers, and tethers, even if fabricated with and/or by the mechanical or electrical component manufacturer, should be designed for seismic forces. This is regardless of whether the mechanical or electrical component itself is designed for seismic loads. The intention is to prevent a component from sliding, falling, toppling, or otherwise moving such that the component would imperil life.

3.3.6 COMPONENT CERTIFICATION: The 1991 *Provisions* imposed a blanket requirement that was of little benefit for most components. Rather than deleting the requirement, it was judged to be beneficial to leave an "advisory" clause to be considered by the engineer-of-record. It is intended that the certificate only be requested for components with an importance factor (I_p) greater than 1.00 and only if the component has a doubtful or uncertain seismic load path. This certificate should not be requested to validate functionality concerns.

In the context of these provisions, seismic adequacy of the component is of concern only when the component is required to remain operational after an earthquake or contains material that can pose a significant hazard if released. Meeting the requirements of this section shall be considered as an acceptable demonstration of the seismic adequacy of a component.

3.3.7 UTILITY AND SERVICE LINES AT BUILDING INTERFACES: For essential facilities, auxiliary on-site mechanical and electrical 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. 3.3.7 requires that adequate flexibility be provided for utilities at the interface of adjacent and independent structures to accommodate anticipated differential displacement. It affects architectural and mechanical/electrical fittings only where water and energy lines pass through the interface. The displacements considered must include the C_d factor of Sec. 2.2.2.

Consideration may be necessary for nonessential piping carrying quantities of materials that could, if the piping is ruptured, damage nearby essential utilities.

3.3.9 STORAGE TANKS: All large storage tanks, all tanks with hazardous contents, or tanks required to remain operable after an earthquake should be designed for seismic forces. At-grade storage tanks should be designed to accommodate piping connection displacements. The design for seismic forces in accordance with industry accepted standards is encouraged. Displacement considerations for connections are based on performance of tanks subject to large earthquake motions.

Performance of flat bottom storage tanks in past earthquakes has indicated that sloshing of contents can cause leakage and roof damage. This damage can be prevented or significantly mitigated by providing freeboard which is greater than the calculated slosh height, d, or by designing the roof and wall connection for the sloshing wave forces. The sloshing height of the tank can be determined by the procedure noted in the reference by Housner and Haroun, summarized as follows:

$$\delta_s = 0.837 R S_a \qquad \text{(C3.3.9-1)}$$

where:

δ_s = slosh height (feet or meters),

R = tank radius (feet or meters),

S_a = spectral acceleration, including site factor, as a multiplier of gravity (e.g., 0.40) corresponding to the sloshing period, T_{slosh} and 0.5 percent damping, as defined below:

$$T_{slosh} = 2\pi \sqrt{\frac{R}{1.84 g \tanh\left(\frac{1.84H}{R}\right)}} \qquad \text{(C3.3.9-2)}$$

where H = liquid height (feet or meters) and g = acceleration of gravity in consistent units (feet or meters).

For T_{slosh} less than 4.5 seconds:

$$S_a = \frac{1.5 C_v}{T_{slosh}} \qquad \text{(C3.3.9-3)}$$

where the 1.5 factor accounts for the additional amplification associated with 0.5 percent damping, rather than 5 percent damping.

For T_{slosh} greater than 4.5 seconds:

$$S_a = \frac{C_v}{3(T_{slosh})^2} \qquad \text{(C3.3.9-4)}$$

3.3.10 HVAC DUCTWORK: Experience in past earthquakes has shown that, in general, HVAC duct systems generally are rugged and perform well in strong shaking motions. Bracing in accordance with the Sheet Metal and Air Conditioning Contractors National Association (SMACNA) seismic bracing guidelines of Ref. 3-14, 3-15, and 3-16 has been shown to be effective in limiting damage to duct systems under earthquake loads. Typical failures have affected system function only and major damage or collapse has been uncommon. The current practice of designing and installing duct systems generally does not address seismic design of the system but results in adequate seismic performance for the majority of duct systems. Industry standard practices should prove adequate for most installations. Expected earthquake damage should be limited to opening of the duct joints and tears in the ducts. Connection details that are prone to brittle failures, especially hanger rods subject to high cyclical bending stress cycles, should be avoided.

Some ductwork systems carry hazardous materials or must remain operational during and after an earthquake. These ductwork system would be designated as having an I_p = 1.5. A detailed engineering analysis for these systems should be performed.

All equipment (e.g., fans, humidifiers, and heat exchangers) attached to the ducts and weighing more than 75 lb (334 N) should be braced independently of the duct. Unbraced in-line equipment can damage the duct by swinging and impacting it during an earthquake. Items (e.g., dampers, louvers, and air diffusers) attached to the duct should be positively supported by mechanical fasteners (not friction-type connections) to prevent their falling during an earthquake.

Where it is desirable to limit the deflection of duct systems under seismic load, bracing in accordance with the SMACNA seismic bracing guidelines listed in Sec. 3.1.1 may be used.

3.3.11 PIPING SYSTEMS: Experience in past earthquakes has shown that, in general, piping systems are rugged and perform well in strong shaking motions. Construction in accordance with current provisions of the ASME B31, *Code for Pressure Piping* (Ref. 3-3); MSS SP-58, *Pipe Hangers and Supports--Materials, Design, and Manufacture* (Ref. 3-11); and NFPA 13 *Standard for Installation of Sprinkler Systems* (Ref 3-12) have been shown to be effective in limiting damage to and avoiding loss of fluid containment in piping systems under earthquake conditions. It is therefore the intention of the *Provisions* that nationally recognized codes be used to design piping systems, provided that the force and displacement demand is equal to or exceeds the provisions of Sec. 3.1.3 and 3.1.4. Until such nationally recognized codes incorporate force and displacement provisions comparable to the provisions of Sec. 3.1.3 and 3.1.4, it is nonetheless the intention to use the design acceptance criteria and construction practices of those codes.

Piping, as used herein, are assemblies of pipe, tubing, valves, fittings, and other in-line fluid containing components, excluding their attachments and supports.

3.3.12 BOILERS AND PRESSURE VESSELS: Experience in past earthquakes has shown that, in general, boilers and pressure vessels are rugged and perform well in strong shaking motions. Construction in accordance with current provisions of the ASME *Boiler and Pressure Vessel Code* (Ref 3-4) has been shown to be effective in limiting damage to and avoiding loss of fluid containment in boilers and pressure vessels under earthquake conditions. It is therefore the intention of the *Provisions* that nationally recognized codes be used to design boilers and pressure vessels provided that the seismic force and displacement demand is equal to or exceeds the provisions of Sec. 3.1.3 and 3.1.4. Until such nationally recognized codes incorporate force and displacement provisions comparable to the provisions of Sec. 3.1.3 and 3.1.4, it is nonetheless the intention to use the design acceptance criteria and construction practices of those codes.

Boilers and pressure vessels as used herein are fired or unfired containments, including their internal and external appurtenances and internal assemblies of pipe, tubing, and fittings, and other fluid containing components, excluding their attachments and supports.

3.3.13 MECHANICAL EQUIPMENT ATTACHMENTS AND SUPPORTS: Past earthquakes have demonstrated that most mechanical equipment is inherently rugged and performs well provided that it is properly attached to the structure. This is because the design of mechanical equipment items for operational and transportation loads typically envelopes loads due to earthquake.

The *Provisions* primarily focus on equipment structural integrity and stability. However, reliability of equipment operability after an earthquake can be increased if the following items are also considered in design:

a. Internal assemblies are attached with a sufficiency that eliminates the potential of impact with other internal assemblies and the equipment wall; and

b. Operators, motors, generators, and other such components functionally attached mechanical equipment by means of an operating shaft or mechanism are structurally connected or commonly supported with sufficient rigidity such that binding of the operating shaft will be avoided.

3.3.14 ELECTRICAL EQUIPMENT ATTACHMENTS AND SUPPORTS: Past earthquakes have demonstrated that most electrical equipment is inherently rugged and performs well provided that it is properly attached to the structure. This is because the design of electrical equipment items for operational and transportation loads typically envelopes loads due to earthquake.

The *Provisions* primarily focus on equipment structural integrity and stability. However, reliability of equipment operability after an earthquake can be increased if the following items also are considered in design:

a. Internal assemblies are attached with a sufficiency that electrical subassemblies and contacts will not be subject to differential movement or impact between the assemblies, contacts, and the equipment enclosure.

b. Any ceramic or other nonductile components in the seismic load path should be specifically evaluated.

c. Adjacent electrical cabinets are bolted together and cabinet lineups are prevented from banging into adjacent structural members.

3.3.15 ALTERNATE SEISMIC QUALIFICATION METHODS: Testing is a well established alternative method of seismic qualification for small to medium size equipment. Several national standards, other than IEEE 344 (Ref. 3-10), have testing provisions adaptable for seismic qualification.

3.3.16 ELEVATOR DESIGN REQUIREMENTS: The ASME *Safety Code for Elevators and Escalators* (Ref. 3-2) has adopted many provisions to improve the seismic response of elevators; however, they do not apply to some regions covered by this chapter. These changes are to extend force requirements for elevators to be consistent with the *Provisions*.

3.3.16.2 Elevator Machinery and Controller Supports and Attachments: The ASME *Safety Code for Elevators and Escalators* (Ref. 3-2) has no seismic provisions for supports and attachments for some structures and zones where the *Provisions* are applicable. Criteria are provided to extend force requirements for elevators to be consistent with the intent and scope of the *Provisions*.

3.3.16.3 Seismic Controls: The purpose of the seismic switch as used here is different from that provided under the ASME *Safety Code for Elevators and Escalators* (Ref. 3-2), which has incorporated several provisions to improve the seismic response of elevators (e.g., rope snag point guard, rope retainer guards, guide rail brackets) that do not apply to some buildings and zones covered by the *Provisions*. Building motions that are expected in these uncovered seismic zones are sufficiently large to impair the operation of elevators. The seismic switch is positioned high in the structure where structural response will be the most severe. The seismic switch trigger level is set to shut down the elevator when structural motions are expected to impair elevator operations.

Elevators in which the seismic switch and counterweight derail device have triggered should not be put back into service without a complete inspection. However, in the case where the loss of use of the elevator creates a life-safety hazard, an attempt to put the elevator back into service may be attempted. Operating the elevator prior to inspection may cause severe damage to the elevator or its components.

The building owner should have detailed written procedures in place directing the elevator operator/maintenance personnel which elevators in the facility are necessary from a post-earthquake life safety perspective. It is highly recommended that these procedures be in-place, with appropriate personnel training prior to an event strong enough to trip the seismic switch.

Once the elevator seismic switch is reset, it will respond to any call at any floor. It is important that the detailed procedure include the posting of "out-of-service for testing" signs at each door at each floor, prior to resetting the switch. Once the testing is completed, and the elevator operator/maintenance personnel are satisfied that the elevator is safe to operate, the signs can be removed.

3.3.16.4 Retainer Plates: The use of retainer plates is a very low cost provision to improve the seismic response of elevators.

RELATED CONCERNS:

Maintenance: Mechanical and electrical devices installed to satisfy the requirements of these provisions (e.g., resilient mounting components or certain protecting devices) require maintenance to ensure their reliability and provide the protection in case of a seismic event for which they are designed. Specifically, rubber-in-shear mounts or spring mounts (if exposed to weathering) may 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.

Tenant Improvements: It is intended that the provisions in Chapter 3 also apply to newly constructed tenant improvements that are listed in Tables 3.2.2 and 3.3.2 and that are installed at any time during the life of the structure.

Minimum Standards: Criteria represented in the provisions represent minimum standards. They are designed to minimize hazard for occupants and to improve the likelihood of 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 structures 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 also may be in short demand since available labor forces may be diverted to high priority structures requiring repairs.

Architect-Engineer Design Integration: The subject of architect-engineer design integration is being raised because it is believed that all members of the profession should clearly understand that Chapter 3 is a compromise based on concerns for enforcement and the need to develop 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.

REFERENCES

Bachman, R. E., R. M. Drake, and P. J. Richter. 1993. *1994 Update to 1991 NEHRP Provisions for Architectural, Mechanical, and Electrical Components and Systems*, letter report to the National Center for Earthquake Engineering Research, February 22.

Housner, G. W., and M. A. Haroun. 1980. "Seismic Design of Liquid Storage Tanks" in *ASCE Convention Proceedings.*

Chapter 4 Commentary

FOUNDATION DESIGN REQUIREMENTS

4.1 GENERAL: The minimum foundation design requirements that might be suitable when any consideration must be given to earthquake resistance are set forth in Chapter 4. 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.

4.2 STRENGTH OF COMPONENTS AND FOUNDATIONS: The resisting capacities of the foundations must meet the provisions of Chapter 4.

4.2.1 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 5, 6, 7, 8, or 9.

4.2.2 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.

4.3 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.

4.4 SEISMIC PERFORMANCE CATEGORY C: Extra precautions are required for the seismic design of foundations for buildings assigned to Category C.

4.4.1 INVESTIGATION: Potential site hazards such as fault rupture, liquefaction, ground deformation, and slope instability should be investigated when the size and importance of the project so warrants. In this section, procedures for evaluating these hazards are reviewed.

Surface Fault Rupture: Fault ruptures during past earthquakes have led to large surface displacements that are potentially destructive to engineered construction. Displacements, which range from a fraction of an inch to tens of feet, generally occur along traces of previously active faults. The sense of displacement ranges from horizontal strike-slip to vertical dip-slip to many combinations of these components. The following commentary summarizes procedures to follow or consider when assessing the hazard of surface fault rupture. This commentary is based in large part on Appendix C of California Division of Mines and Geology (CDMG) Special Publication 42, 1988 Revision (Hart, 1988).

Assessment of Surface Faulting Hazard: The evaluation of fault hazard at a given site is based extensively on the concepts of recency and recurrence of faulting along existing faults. The magnitude, sense, and frequency of fault rupture vary for different faults or even along different segments of the same fault. Even so, future faulting generally is expected to recur along pre-existing faults. The development of a new fault or reactivation of a long inactive fault is relatively uncommon and generally need not be a concern. For most engineering applications, a sufficient definition of an active fault is given in CDMG Special Publication 42 (Hart, 1988): "An active fault has had displacement in Holocene time (last 11,000 years)."

As a practical matter, fault investigations should be conducted by qualified geologists and directed at the problem of locating faults and evaluating recency of activity, fault length, and the amount and character of past displacements. Identification and characterization studies should incorporate evaluation of regional fault patterns as well as detailed study of fault features at and in the near vicinity (within a few hundred yards to a mile) of the site. Detailed studies should include trenching to accurately locate, document, and date fault features.

Suggested Approach for Assessing Surface Faulting Hazard: The following approach should be used, or at least considered, in fault hazard assessment. Some of the investigative methods outlined below should be carried out beyond the site being investigated. However, it is not expected that all of the following methods would be used in a single investigation:

1. A review should be made of the published and unpublished geologic literature from the region along with records concerning geologic units, faults, ground-water barriers, etc.

2. A stereoscopic study of aerial photographs and other remotely sensed images should be made to detect fault-related topography, vegetation and soil contrasts, and other lineaments of possible fault origin. Predevelopment air photos are essential to the detection of fault features.

3. A field reconnaissance study generally is required which includes observation and mapping of geologic and soil units and structures, geomorphic features, springs, and deformation of man-made structures due to fault creep. This study should be detailed within the site with less detailed reconnaissance of an area within a mile or so of the site.

4. Subsurface investigations usually are needed to evaluate fault features. These investigations include trenches, pits, or bore holes to permit detailed and direct observation of geologic units and fault features.

5. The geometry of fault structures may be further defined by geophysical investigations including seismic refraction, seismic reflection, gravity, magnetic intensity, resistivity, ground penetrating radar, etc. These indirect methods require a knowledge of specific geologic conditions for reliable interpretation. Geophysical methods alone **never** prove the absence of a fault and they do not identify the recency of activity.

6. More sophisticated and more costly studies may provide valuable data where geological special conditions exist or where requirements for critical structures demand a more intensive investigation. These methods might involve repeated geodetic surveys, strain measurements, or monitoring of microseismicity and radiometric analysis (^{14}C, K-Ar), stratigraphic correlation (fossils, mineralology) soil profile development, paleomagnetism (magneto-stratigraphy), or other age-dating techniques to date the age of faulted or unfaulted units or surfaces.

The following information should be developed to provide documented support for conclusions relative to location and magnitude of faulting hazards:

1. Maps should be prepared showing the existence (or absence) and location of hazardous faults on or near the site.

2. The type, amount, and sense of displacement of past surface faulting episodes should be documented including sense and magnitude of displacement, if possible.

3. From this documentation, estimates can be made, preferably from measurements of past surface faulting events at the site, using the premise that the general pattern of past activity will repeat in the future. Estimates also may be made from empirical correlations between fault displacement and fault length or earthquake magnitude published by Bonilla et al. (1984) or by Slemmons et al. (1989). Where fault segment length and sense of displacement are defined, these correlations may provide an estimate of future fault displacement (either the maximum or the average to be expected).

There are no codified procedures for estimating the amount or probability of future fault displacements. Estimates may be made, however, by qualified earth scientists. Because techniques for making these estimates are not standardized, peer review of reports is useful to verify the adequacy of the methods used and the estimates reports, to aid the evaluation by the permitting agency, and to facilitate discussion between specialists that could lead to the development of standards.

The following guidelines are given for safe siting of engineered construction in areas crossed by active faults:

1. Where ordinances have been developed that specify safe setback distances from traces of active faults or active fault zones, those distances must be complied with and accepted as the minimum for safe siting of buildings. For example, the general setback requirement in California is a minimum of 50 feet from a well-defined zone containing the traces of an active fault. That setback distance is mandated as a minimum for structures near faults unless a site-specific special geologic investigation shows that a lesser distance could be safety applied (California Administrative Code, Title 14, Sec. 3603A).

2. In general, safe setback distances may be determined from geologic studies and analyses as noted above. Setback requirements for a site should be developed by the site engineers and geologists in consultation with professionals from the building and planning departments of the jurisdiction involved. Where sufficient geologic data have been developed to accurately

locate the zone containing active fault traces and the zone is not complex, a 50-foot setback distance may be specified. For complex fault zones, greater setback distances may be required. Dip-slip faults, with either normal or reverse motion, typically produce multiple fractures within rather wide and irregular fault zones. These zones generally are confined to the hanging-wall side of the fault leaving the footwall side little disturbed. Setback requirements for such faults may be rather narrow on the footwall side, depending on the quality of the data available, and larger on the hanging wall side of the zone. Some fault zones may contain broad deformational features such as pressure ridges and sags rather than clearly defined fault scarps or shear zones. Nonessential structures may be sited in these zones provided structural mitigative measures are applied as noted below. Studies by qualified geologists and engineers are required for such zones to assure that building foundations can withstand probable ground deformations in such zones.

Mitigation of Surface Faulting Hazards: There is no mitigative technology that can be used to prevent fault rupture from occurring. Thus, sites with unacceptable faulting hazard must either be avoided or structures designed to withstand ground deformation or surface fault rupture. In general practice, it is economically impractical to design a structure to withstand more than a few inches of fault displacement. Some buildings with strong foundations, however, have successfully withstood or diverted a few inches of surface fault rupture without damage to the structure (Youd, 1989). Well reinforced mat foundations and strongly inter-tied footings have been most effective. In general, less damage has been inflicted by compressional or shear displacement than by vertical or extensional displacements.

Liquefaction: Liquefaction of saturated granular soils has been a major source of building damage during past earthquakes. For example, many structures in Niigata, Japan, suffered major damage as a consequence of liquefaction during the 1964 earthquake. Loss of bearing strength, differential settlement, and differential horizontal displacement due to lateral spread were the direct causes of damage. Many structures have been similarly damaged by differential ground displacements during U.S. earthquakes such as the San Fernando Valley Juvenile Hall during the 1971 San Fernando, California, earthquake and the Marine Sciences Laboratory at Moss Landing, California, during the 1989 Loma Prieta event. Design to prevent damage due to liquefaction consists of three parts: evaluation of liquefaction hazard, evaluation of potential ground displacement, and mitigating the hazard by designing to resist ground displacement, by reducing the potential for liquefaction, or by choosing an alternative site with less hazard.

Evaluation of Liquefaction Hazard: Liquefaction hazard at a site is commonly expressed in terms of a factor of safety. This factor is defined as the ratio between the available liquefaction resistance, expressed in terms of the cyclic stresses required to cause liquefaction, and the cyclic stresses generated by the design earthquake. Both of these stress parameters are commonly normalized with respect to the effective overburden stress at the depth in question.

The following possible methods for calculating the factor of safety against liquefaction have been proposed and used to various extents:

1. Analytical Methods -- These methods typically rely on laboratory test results to determine either liquefaction resistance or soil properties that can be used to predict the development of liquefaction. Various equivalent linear and nonlinear computer methods are used with the laboratory data to evaluate the potential for liquefaction. Because of the considerable difficulty in obtaining undisturbed samples of liquefiable sediment for laboratory evaluation

of constitutive soil properties, the use of analytical methods, which rely on accurate constitutive properties, usually are limited to critical projects or to research.

2. Physical Modeling -- These methods typically involve the use of centrifuges or shaking tables to simulate seismic loading under well defined boundary conditions. Soil used in the model is reconstituted to represent different density and geometrical conditions. Because of difficulties in precisely modeling in-situ conditions at liquefiable sites, physical models have seldom been used in design studies for specific sites. However, physical models are valuable for analyzing and understanding generalized soil behavior and for evaluating the validity of constitutive models under well defined boundary conditions.

3. Empirical Procedures -- Because of the difficulties in analytically or physically modeling soil conditions at liquefiable sites, empirical methods have become a standard procedure for determining liquefaction susceptibility in engineering practice. Procedures for carrying out a liquefaction assessment using the empirical method are given by the National Research Council (1985).

For most empirical methods, the average earthquake-induced cyclic shear stress is estimated from a simple equation or from dynamic response analyses using computer programs such as SHAKE and DESRA. The induced cyclic shear stress is estimated from the peak horizontal acceleration expected at the site using the following simple equation:

$$\frac{\tau}{\sigma_o'} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_o}{\sigma_o'}\right) r_d \qquad \text{(C4.4.1-1)}$$

where:

(a_{max}/g)	=	peak horizontal acceleration at ground surface expressed as a decimal fraction of gravity,
σ_o	=	the vertical total stress in the soil at the depth in question,
σ_o'	=	the vertical effective stress at the same depth, and
r_d	=	deformation-related stress reduction factor.

The chart reproduced in Figure C4.4.1-1 is used to estimate r_d.

To determine liquefaction resistance of sandy soils, the induced cyclic stress ratio computed from Eq. C4.4.1-1 is compared to the cyclic stress ratio required to generate liquefaction in the soil in question for a given earthquake of magnitude M. The most common technique for estimating liquefaction resistance is from an empirical relationship between cyclic stress ratio required to cause liquefaction and normalized blow count, $(N_1)_{60}$.

The most commonly used empirical relationship, compiled by Seed et al. (1985), compares $(N_1)_{60}$ from sites where liquefaction did or did not develop during past earthquakes. Figure C4.4.1-2 shows the most recent (1988) version of this relationship for M = 7-1/2 earthquakes.

On that figure, cyclic stress ratios calculated for various sites are plotted against $(N_1)_{60}$. Solid dots represent sites where liquefaction occurred and open dots represent sites where surface evidence of liquefaction was not found. Curves were drawn through the data to separate regions where liquefaction did and did not develop. Curves are given for sediments with various fines contents.

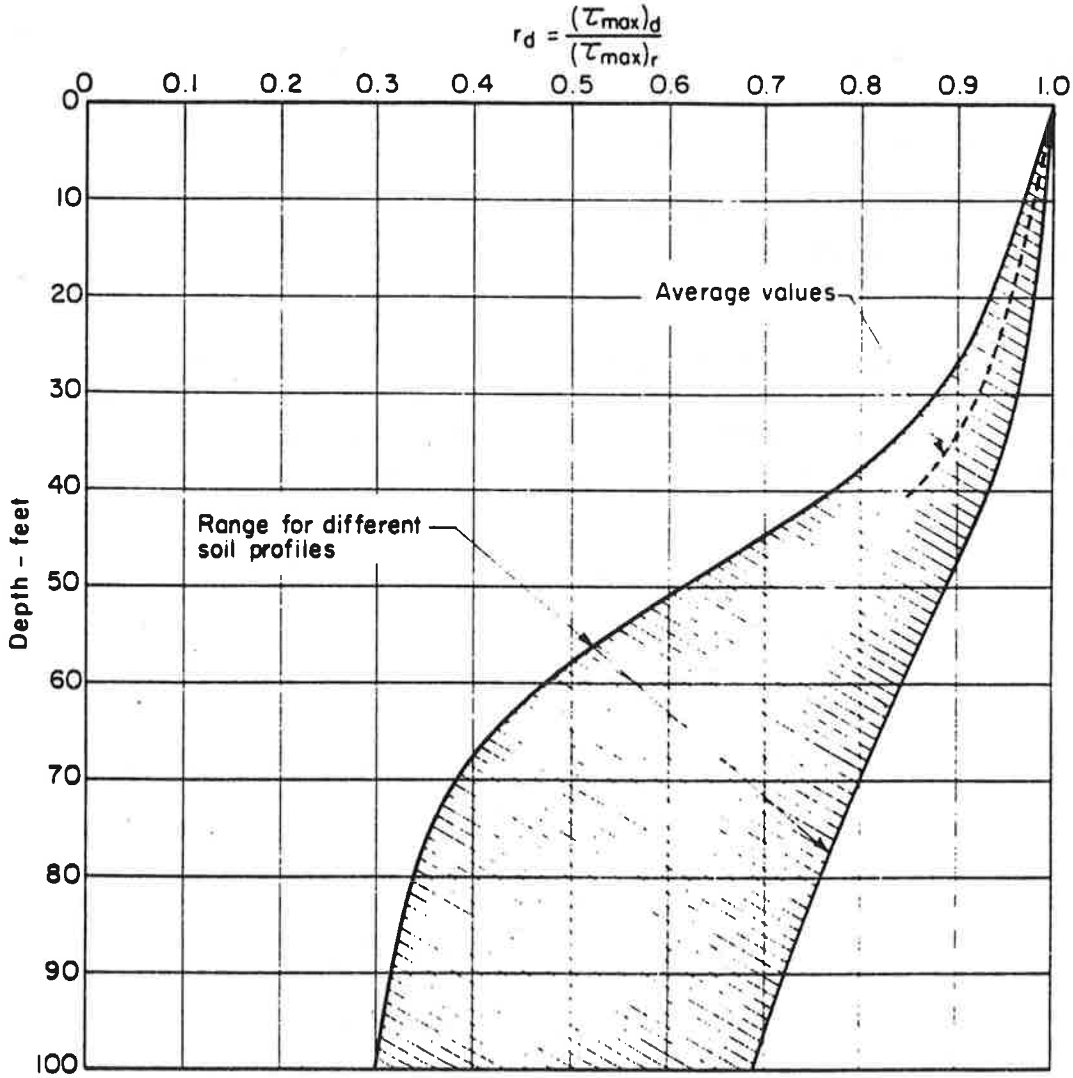


FIGURE C4.4.1-1 Range of values for r_d for different soil properties (after Seed and Idriss, 1971).

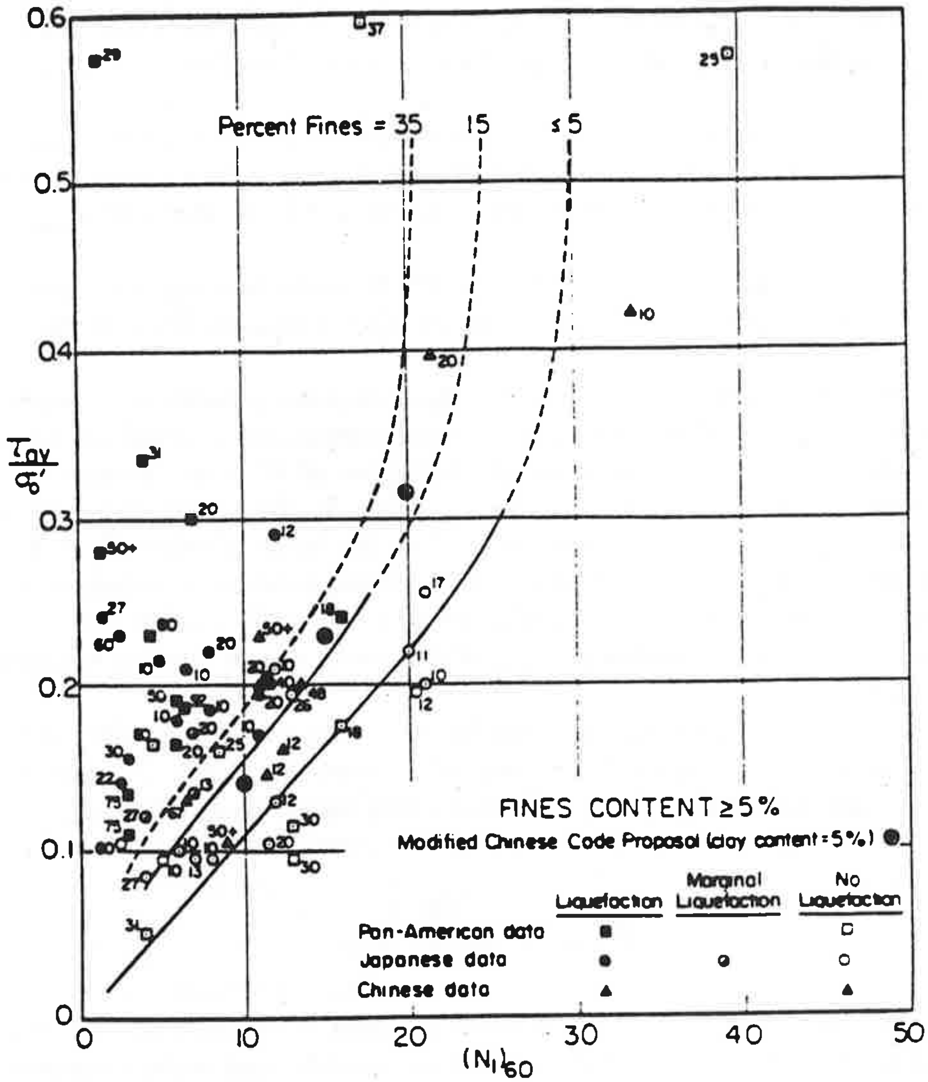


FIGURE C4.4.1-2 Relationship between stress ratios causing liquefaction and N_1 values for silty sands for M = 7-1/2 earthquakes.

Although the curves drawn by Seed et al. (1985) envelop the plotted data, it is possible that liquefaction may have occurred beyond the enveloped data and was not detected at ground surface. Consequently, a factor of safety of 1.2 to 1.5 is appropriate in engineering design. The factor to be used is based on engineering judgment with appropriate consideration given to type and importance of structure and potential for ground deformation.

The maximum acceleration, a_{max}, commonly used in liquefaction analysis is that which would occur at the site in the absence of liquefaction. Thus, the a_{max} used in Eq. C4.4.1-1 is the estimated rock acceleration corrected for soil site response but with neglect of excess pore-water pressures that might develop. Alternatives for obtaining a_{max} are:

1. From standard peak acceleration attenuation curves valid for comparable soil conditions;

2. From standard peak acceleration attenuation curves for rock, corrected for site amplification or deamplification by means of standard amplification curves or computerized site response analysis such as described in the "Chapter 1 Commentary" for Sec. 1.4.2;

3. Obtaining first the value of effective peak acceleration, A_a, for rock depending on the map area where the site is located and then multiplying this value by a factor between 1 and 3 as discussed in the "Chapter 1 Commentary" for Sec. 1.4.2 to determine a_{max}.

4. From probabilistic maps of a_{max} with or without correction for site amplification or deamplification depending on the rock or soil conditions used to generate the map.

The magnitude, M, needed to determine a magnitude scaling factor from Figure C4.4.1-3 should correspond to the size of the design or expected earthquake selected for the liquefaction evaluation. If Alternative 3 or 4 is selected, the definition of M is not obvious and additional studies and considerations are necessary. In all cases, it should be remembered that the likelihood of liquefaction at the site (as defined later by the factor of safety F_L in Eq. C4.4.1-3) is determined **jointly** by a_{max} and M. Because of the longer duration of strong ground-shaking, large distant earthquakes may generate liquefaction at a site while smaller nearby earthquakes may not generate liquefaction even though a_{max} of the nearer events is larger than that from the more distant events.

The corrected blow count, $(N_1)_{60}$, required for evaluation of soil liquefaction resistance is commonly determined from measured standard penetration resistance, N_m, but may also be determined from cone penetration test (CPT) data using standard correlations to estimate N_m values from the CPT measurements. The corrected blow count is calculated from N_m as follows:

$$(N_1)_{60} = C_n\left(\frac{ER_m}{60}\right)N_m \tag{C4.4.1-2}$$

where C_n = a factor that corrects N_m to an effective overburden pressure of 1 tsf and ER_m = the rod energy ratio for the type of hammer and release mechanism used in the measurement of N_m.

The curve plotted in Figure C4.4.1-4 is typically used to evaluate C_n. Measured hammer energies or estimates of hammer energies from tabulations such as those in Table C4.4.1 are used to define ER_m. An additional correction should be made to $(N_1)_{60}$ for shallow soil layers where the length of drilling rod is 10 feet or less. In those instances, $(N_1)_{60}$ should be reduced by multiplying by a factor of 0.75 to account for poor hammer-energy transfer in such short rod lengths.

Because a variety of equipment and procedures are used to conduct standard penetration tests in present practice and because the measured blow count, N_m, is sensitive to the equipment and procedures used, the following commentary and guidance with respect to this test is given. Special attention must be paid to the determination of normalized blow count, $(N_1)_{60}$, used in Figure C4.4.1-2. When developing the empirical relation between blow count and liquefaction resistance, Seed and his colleagues recognized that the blow count from SPT is greatly influenced by factors such as the method of drilling, the type of hammer, the sampler design, and the type of mechanism used for lifting and dropping the hammer. The magnitude of variations is shown by the data in Table C4.4.1.

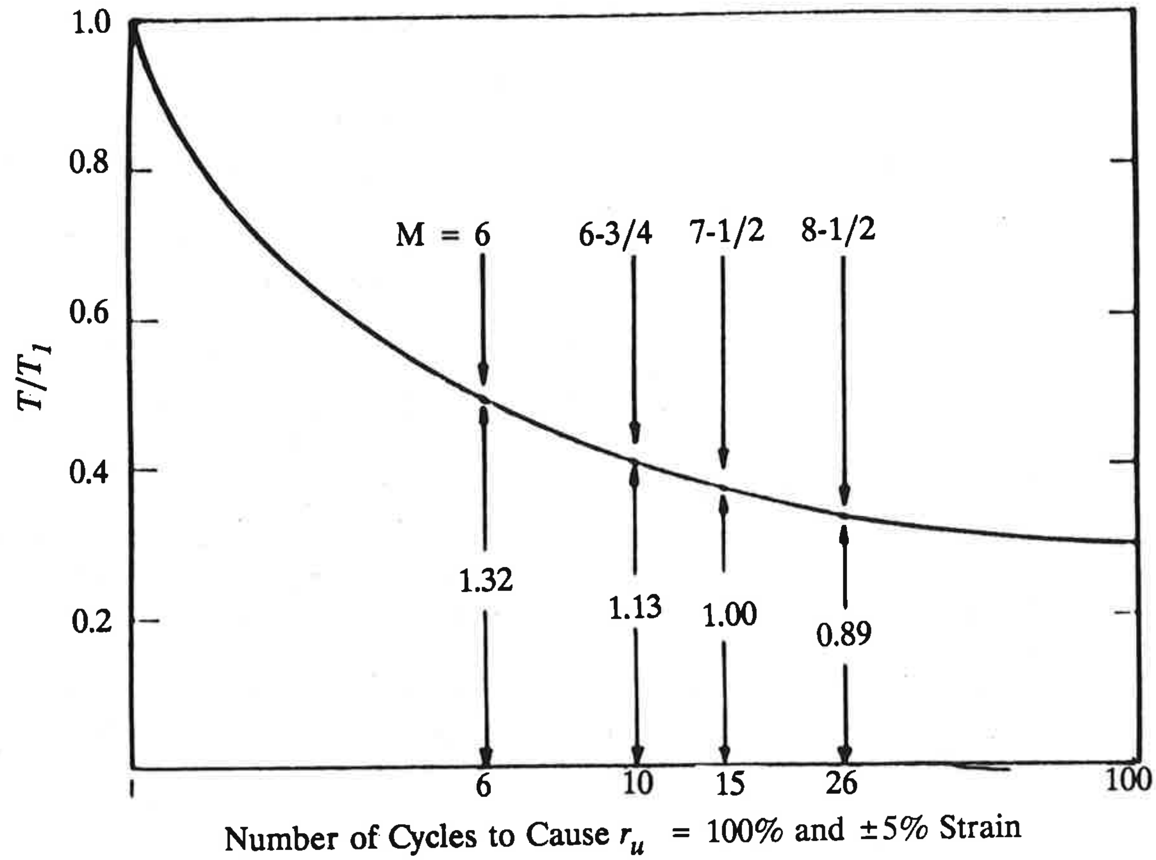


FIGURE C4.4.1-3 Representative relationship between T/T_1 and number of cycles required to cause liquefaction (after Seed et al., 1983).

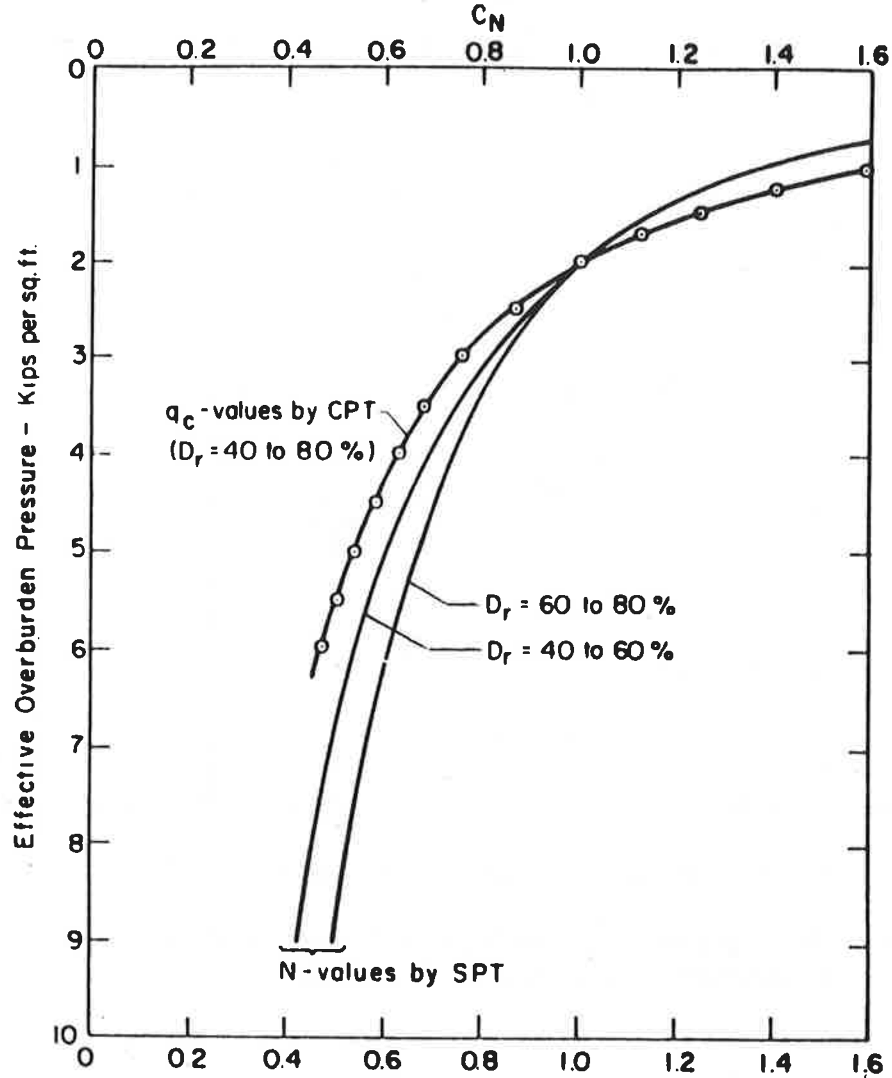


FIGURE C4.4.1-4 Chart for C_n (after Seed et al., 1985).

TABLE C4.4.1
Summary of Rod Energy Ratios for Japanese SPT Procedures
(After Seed et al., 1985)

Study	Mechanical Trip System (Tonbi)	Rope and Pulley
Nishizawa et al.	80-90	63-72
Decker, Holtz, and Kovacs	76	--
Kovacs and Salomone	80	67
Tokimatsu and Yoshimi	76[a]	--
Yoshimi and Tokimatsu, Yoshimi et al., Oh-Oka	--	--
Adopted for this study	78	67

[a] Equivalent rod energy ratio if rope and pulley method is assumed to have an energy ratio of 67 percent and values for mechanical trip method are different from this by a factor of 1.13.

In order to reduce variability in the measurement of *N*, Seed et al. (1983 and 1985) suggest the following procedures and specifications for the SPT test for liquefaction investigations:

1. The impact should be delivered by a rope and drum system with two turns of the rope around the rotating drum to lift a hammer weighing 140 lb or, more preferably, a drive system should be used for which ER_m has been measured or can be reliably estimated.

2. Use of a hole drilled with rotary equipment and filled with drilling mud. The hole should be approximately 4 in. in diameter and drilled with a tricone or baffled drag bit that produces upward deflection of the drilling fluid to prevent erosion of soil below the cutting edge of the bit.

3. In holes less than 50 feet deep, A or AW rod should be used; N or NW rod should be used in deeper holes.

4. The split spoon sampling tube should be equipped with liners or otherwise have a constant internal diameter of 1-3/8 inch.

5. Application of blows should be at a rate of 30 to 40 blows per minutes. (Some engineers suggest a slower rate of 20 to 30 blows per minute since it is easier to achieve and control and gives comparable results.) The blow count, N_m, is determined by counting the blows required to drive the penetrometer through the depth interval of 6 to 18 in. below the bottom of the hole.

Failure to follow these standard guidelines introduces large uncertainties into liquefaction estimates.

The curves in Figure C4.4.1-2 were developed from data for magnitude 7.5 earthquakes and are only valid for earthquakes of that magnitude. For larger or smaller earthquakes, the cyclic stress ratios determined from Figure C4.4.1-2 are corrected for magnitude by multiplying the determined cyclic stress ratio by a magnitude scaling factor taken from Figure C4.4.1-3. As the magnitude increases, the scaling factor decreases. For example, for an $(N_1)_{60}$ of 20, a clean sand (fines content < 5 percent) and an earthquake magnitude of 7.5, the CSRL determined from Figure C4.4.1-2 is 0.22. For the same site conditions but for a magnitude 8.0 earthquake, a CSRL of 0.20 is obtained after applying the magnitude scaling factor of 0.89 determined from Figure C4.4.1-3.

Soils composed of sands, silts, and gravels are most susceptible to liquefaction while clayey soils generally are immune to this phenomenon. The curves in Figure C4.4.1-2 are valid for soils composed primarily of sand. The curves should be used with caution for soils with substantial amounts of gravel. Verified corrections for gravel content have not been developed; a geotechnical engineer, experienced in liquefaction hazard evaluation, should be consulted when gravelly soils are encountered. For soils containing more than 35 percent fines, the curve in Figure C4.4.1-2 for 35 percent fines should be used provided the following criteria developed by Seed et al. (1983) are met (i.e, the weight of soil particles finer than 0.005 mm is less than 15 percent of the dry weight of a specimen of the soil, the liquid limit of soil is less than 35 percent, and the moisture content of the in-place soil is greater than 0.9 times the liquid limit.

In summary, the procedure for evaluation of liquefaction resistance for a site is as follows: First, from a site investigation determine the measured standard penetration resistance, N_m, the percent fines, the percent clay (> 0.005 mm), the natural moisture content, and the liquid limit of the sediment in question. Check the measured parameters against the fines content and moisture criteria listed above to assure that the sediment is of a potentially liquefiable type. If so, correct N_m to $(N_1)_{60}$ using Eq. C4.4.1-2 and use Figure C4.4.1-2 to determine the cyclic stress ratio required to cause liquefaction for a magnitude 7.5 earthquake. Then correct that value using the appropriate magnitude scaling factor. That product is the cyclic stress ratio required to cause liquefaction in the field (CSRL). Next, calculate the cyclic stress ratio (CSRE) that would be generated by the expected earthquake using Eq. C4.4.1-1. Then compute the factor of safety, F_L, against liquefaction from the equation:

$$F_L = \frac{CSRL}{CSRE} \qquad \text{(C4.4.1-3)}$$

If F_L is greater than one, then liquefaction should not develop. If at any depth in the sediment profile, F_L is equal to or less than one, then there is a liquefaction hazard. As noted above, a factor of safety of 1.2 to 1.5 is appropriate for building sites with the factor selected depending on the importance of the structure and the potential for ground displacement at the site.

Evaluation of Potential for Ground Displacements: Liquefaction by itself may or may not be of engineering significance. Only when liquefaction is accompanied by loss of ground support and/or ground deformation does this phenomenon become important to structural design. Loss of bearing capacity, flow failure, lateral spread, ground oscillation, and ground settlement are ground failure mechanisms that have caused structural damage during past earthquakes. These types of ground failure are described by the National Research Council (1985). The type of failure and amount of ground displacement are a function of several parameters including the thickness and extent of the liquefied layer, the thickness of unliquefied material overlying the liquefied layer, the ground slope, and the nearness of a free face. Criteria are given by Ishihara (1985) for evaluating the influence of thickness of layers on surface manifestation of liquefaction effects (ground fissures and sand boils) for level sites. These criteria may be used for noncritical or nonessential structures on level sites. Additional analysis should be required for critical or essential structures.

Loss of Bearing Strength: Loss of bearing strength is not likely for light structures with shallow footings founded on stable, nonliquefiable materials overlying deeply buried liquefiable layers, particularly if the liquefiable layers are relatively thin. General guidance for how deep or how thin the layers must be has not yet been developed. A geotechnical engineer, experienced in liquefaction hazard assessment, should be consulted to provide such guidance. Although loss of bearing strength may not be a hazard for deeply buried liquefiable layers, liquefaction-induced ground settlements or lateral-spread displacements could still cause damage and should be evaluated.

Ground Settlement: Tokimatsu and Seed (1987) published an empirical procedure for estimating ground settlement. It is beyond the scope of this commentary to outline that procedure which, although explicit, has several rather complex steps. For saturated or dry granular soils in a loose condition, their analysis suggests that the amount of ground settlement could approach 3 to 4 percent of the thickness of the loose soil layer. The Tokimatsu and Seed technique is recommended for estimating earthquake-induced ground settlement at sites underlain by granular soils and can be applied whether liquefaction does or does not occur.

Horizontal Ground Displacement: Only primitive analytical and empirical techniques have been developed to date to estimate ground displacement, and no single technique has been widely accepted or verified for engineering design. Analytical techniques generally apply Newmark's analysis of a rigid body sliding on an infinite or circular failure surface with ultimate shear resistance estimated from the residual strength of the deforming soil. Alternatively, nonlinear finite element methods have been used to predict deformations. Empirical procedures use correlations between past ground displacement and site conditions under which those displacements occurred. The liquefaction severity index (LSI) correlation of Youd and Perkins (1987) provides a conservative upper bound for displacement for most natural soils (Figure C4.4.1-5; curves noted for various earthquakes are calculated from the equation on the figure). In this procedure, maximum horizontal displacement of lateral spreads in late Holocene fluvial deposits are correlated against earthquake magnitude and distance for the seismic source. The data are from the western United States and the correlation is valid only for that region. Because maximum displacements at very liquefiable sites were used in the LSI analysis, displacements predicted by that technique are conservative in that they predict an upper bound displacement for most natural deposits. Displacements may be greater, however, on uncompacted fill or extremely loose natural deposits.

The following further information is given for general guidance for ground conditions and range of displacements commonly associated with liquefaction-induced ground failures (National Research Council, 1985):

1. Flow failures generally develop in loose saturated sands or silts on slopes greater than 3 degrees (5 percent) and may displace large masses of soil tens of meters. Standard limit equilibrium slope stability analyses may be used to assess flow failure potential with the residual strength used as the strength parameter in the analyses. The residual strength may be determined from empirical correlations such as that published by Seed and Harder (1989).

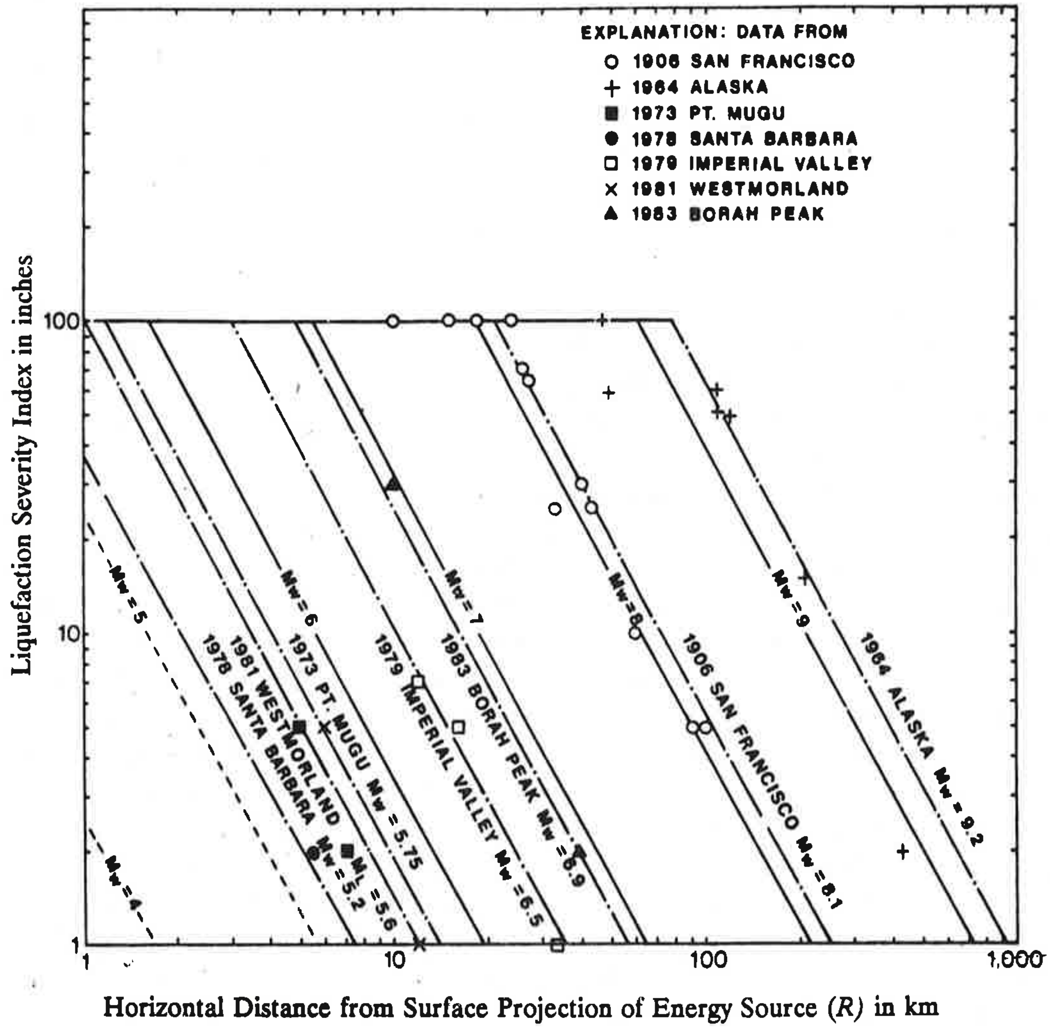


FIGURE 4.4.1-5 LSI from several western U.S. and Alaskan earthquakes plotted against horizontal distance from seismic energy sources (after Youd and Perkins, 1987).

2. Lateral spreads generally develop on gentle slopes between 0.3 and 3 degrees (0.5 and 5 percent) and may induce up to several feet of lateral displacement. The amount of horizontal displacement generally increases with nearness to a free face such as an incised river channel, irrigation or drainage canal, or other open excavation.

3. Ground oscillation occurs on nearly flat surfaces where the slope is too gentle to induce permanent horizontal displacement. During an earthquake, however, ground-shaking generates transient oscillations or ground waves with vertical or horizontal displacements that may range up to a few feet. For example, ground oscillation caused the rather chaotic

pattern of ground displacements that offset pavements, thrust sidewalks over curbs, etc., in San Francisco's Marina District following the 1989 Loma Prieta earthquake.

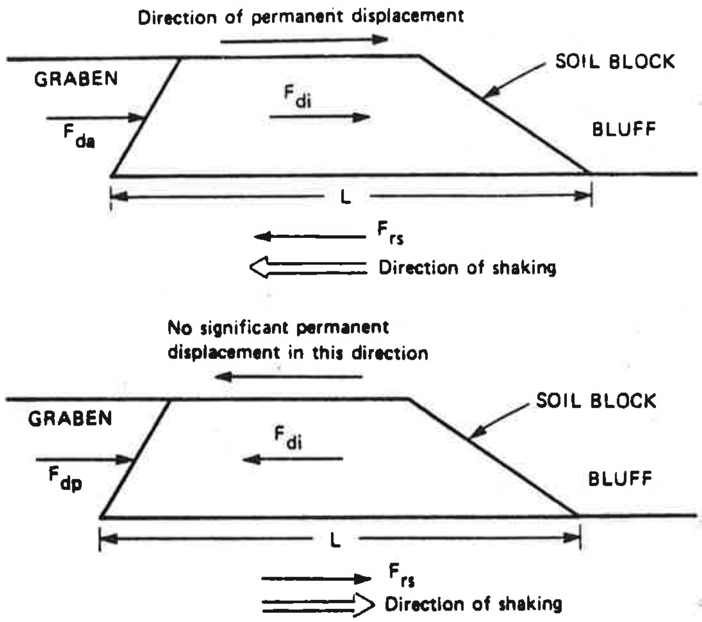
Mitigation of Liquefaction Hazard: With respect to liquefaction hazard, three mitigative measures might be considered: design the structure to resist the hazard, stabilize the site to reduce the hazard, or choose an alternative site. Structural measures that are used to reduce the hazard include deep foundations, mat foundations, or footings interconnected with ties as discussed in Sec. 4.4.3. Deep foundations have performed well at level sites of liquefaction where effects were limited to ground settlement and ground oscillation with no more than a few inches of lateral displacement. Deep foundations, such as piles, may receive very little soil support through the liquefied layer and may be subjected to transient lateral displacements across the layer. Well reinforced mat foundations also have performed well at localities where ground displacements were less than 1 foot although releveling of the structure has been required in some instances (Youd, 1989). Strong ties between footings also should provide increased resistance to damage where differential ground displacements are less than a few inches.

At sites where expected ground displacements are unacceptably large, ground modification to lessen the liquefaction or ground failure hazard or selection of an alternative site may be required. Techniques for ground stabilization to prevent liquefaction of potentially unstable soils include removal and replacement of soil; compaction of soil in place using vibrations, heavy tamping, compaction piles, or compaction grouting; buttressing; chemical stabilization with grout; and installation of drains. Further explanation of these methods is given by the National Research Council (1985).

Slope Instability: The stability of slopes composed of dense (nonliquefiable) or nonsaturated sandy soils or nonsensitive clayey soils can be determined using standard procedures.

For initial evaluation, the pseudostatic analysis may be used. (The deformational analysis described below, however, is now preferred.) In the pseudostatic analysis, inertial forces generated by earthquake shaking are represented by an equivalent static horizontal force acting on the slope. The seismic coefficient for this analysis should be the peak acceleration, a_{max}, or A_a. The factor of safety for a given seismic coefficient can be estimated by using traditional slope stability calculation methods. A factor of safety greater than one indicates that the slope is stable for the given lateral force level and further analysis is not required. A factor of safety of less than one indicates that the slope will yield and slope deformation can be expected and a deformational analysis should be made using the techniques discussed below.

Deformational analyses yielding estimates of slope displacement are now accepted practice. The most common analysis uses the concept of a frictional block sliding on a sloping plane or arc. In this analysis, seismic inertial forces are calculated using a time history of horizontal acceleration as the input motion. Slope movement occurs when the driving forces (gravitational plus inertial) exceed the resisting forces. This approach estimates the cumulative displacement of the sliding mass by integrating increments of movement that occur during periods of time when the driving forces exceed the resisting forces. Displacement or yield occurs when the earthquake ground accelerations exceed the acceleration required to initiate slope movement or yield acceleration. The yield acceleration depends primarily on the strength of the soil and the gradient and height and other geometric attributes of the slope. See Figure C4.4.1-6 for forces and equations used in analysis and Figure C4.4.1-7 for a schematic illustration for a calculation of the displacement of a soil block toward a bluff.



F_{da} = Driving force due to active soil pressure

F_{di} = Driving force due to earthquake inertia

F_{rs} = Resisting force due to soil shear strength

F_{dp} = Resisting force due to passive soil pressure

$$F_{di} = K_{max} W$$

where K_{max} = maximum seismic coefficient

W = weight of soil block

$$F_{rs} = S_u L$$

where S_u = average undrained shear strength of soil

L = length of soil block

Yield Seismic Coefficient

$$K_y = \frac{F_{rs} - F_{da}}{W}$$

FIGURE C4.4.1-6 Forces and equations used in analysis of translatory landslides for calculating permanent lateral displacements from earthquake ground motions (National Research Council, 1985; from Idriss, 1985).

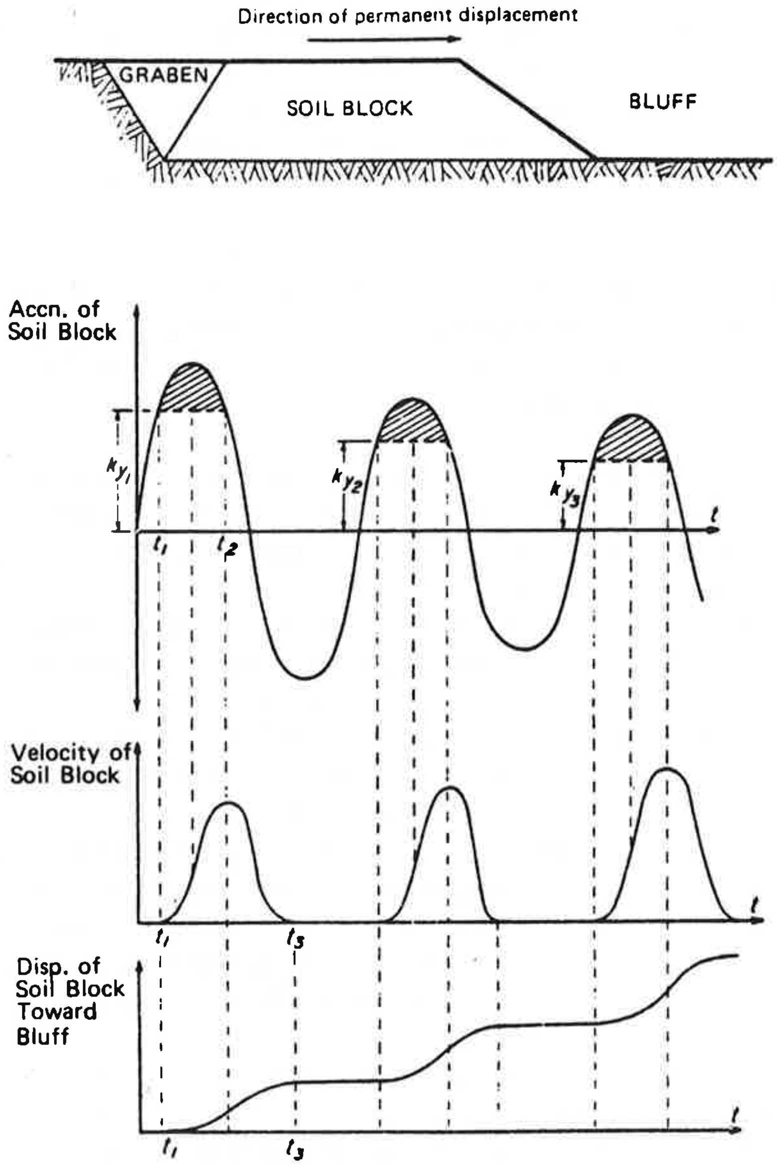


FIGURE C4.4.1-7 Schematic illustration for calculating displacement of soil block toward the bluff (National Research Council, 1985; from Idriss, 1985, adapted from Goodman and Seed, 1966).

The cumulative permanent displacement will depend on the yield acceleration as well as the intensity and duration of ground-shaking. As a general guide, a ratio of yield acceleration to maximum acceleration of 0.5 will result in slope displacements of the order of a few inches for typical magnitude 6.5 earthquakes and perhaps several feet of displacement for magnitude 8 earthquakes. Further guidance on slope displacement is given by Makdisi and Seed (1978).

Mitigation of Slope Instability Hazard: With respect to slope instability, three general mitigative measures might be considered: design the structure to resist the hazard, stabilize the site to reduce the hazard, or choose an alternative site. Ground displacements generated by slope instability are similar in destructive character to fault displacements generating similar senses of movement: compression, shear, extension or vertical. Thus, the general comments on structural design to prevent damage given under mitigation of fault displacement apply equally to slope displacement. Techniques to stabilize a site include reducing the driving forces by grading and drainage of slopes and increasing the resisting forces by subsurface drainage, buttresses, ground anchors, or chemical treatment.

4.4.2 POLE-TYPE STRUCTURES: The use of pole-type structures is permitted. These structures are inherently sensitive to earthquake motions. Bending in the poles and the soil capacity for lateral resistance of the portion of the pole embedded in the ground should be considered and the design completed accordingly.

4.4.3 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 $C_a/4$ 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).

4.4.4 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 common 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 C4.4.4. 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.

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.

4.5 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.

4.5.1 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.

4.5.2 FOUNDATION TIES: The additional requirement is made that spread footings on soft soil profiles should be interconnected by ties. The reasoning explained above under Sec. 4.4.3 also applies here.

4.5.3 SPECIAL PILE REQUIREMENTS: Additional pile reinforcing over that specified for Category C buildings is required. The reasoning explained above under Sec. 4.4.4 applies here.

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.

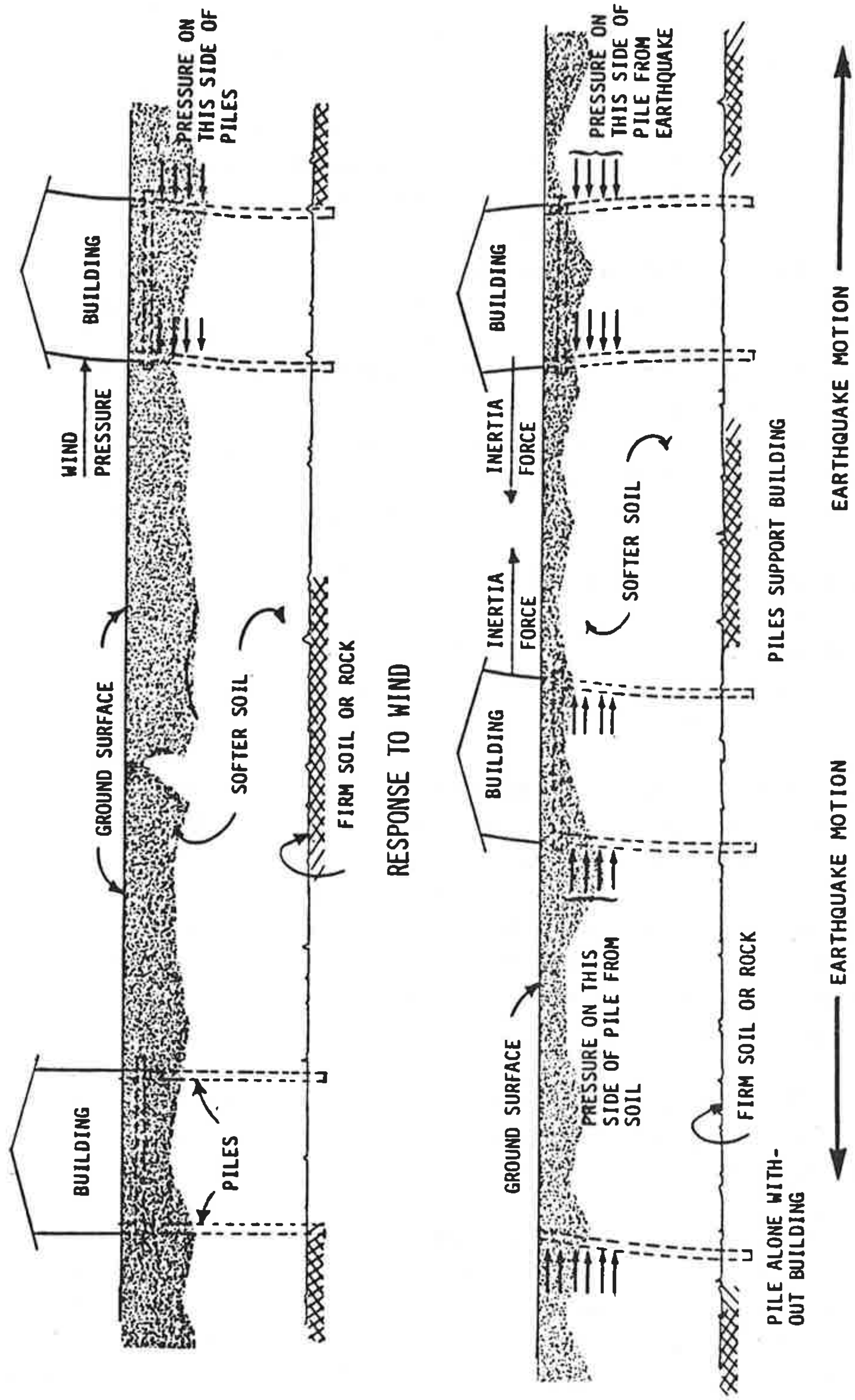


FIGURE C4.4.4 Response to earthquake.

For example:

1. 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.

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

3. 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:

1. Use of a heavy spiral reinforcement and

2. 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.

REFERENCES:

Bertero, V. V, and H. A. Brauer, G. C. Fotinos, G. C. Gerwick, Jr., Y. T. Lin, and H. B. Seed. 1974. "Aseismic Design of Prestressed Concrete Piling." In *Proceedings of the 7th Congress of the Federation Internationale de la Precontrainte*, New York, May 25.

Bieganousky, W. A., and E. F. Marcuson, III. 1976. *Liquefaction Potential of Dams and Foundations: Report 1--Laboratory Standard Penetration Test on Reid Bedford Model and Ottawa Sands,* Report S-76-2. Waterways Experiment Station.

Bieganousky, W. A., and W. F. Marcuson, III. 1977. *Liquefaction Potential of Dams and Foundations: Report 2--Laboratory Standard Penetration Test on Platte River Sand and Standard Concrete Sand,* Report S-762. Waterways Experiment Station.

Bonilla, M. G., R. K. Mark, and J. J. Lienkaemper. 1984. "Statistical Relations Among Earthquake Magnitude, Surface Rupture Length, and Surface Fault Displacement." *Bulletin of the Seismological Society of America* 74(6):2379-2411.

Building Research Institute of Japan. *Report on the Damage by the 1978 Off Miyagi Prefecture Earthquake,* Report 86, pp. 319-341.

California Department of Transportation. 1971. *The San Fernando Earthquake--Field Investigation of Bridge Damage: Preliminary Report.* Sacramento: CALTRANS.

Castro, G. 1975. "Liquefaction and Cyclic Mobility of Saturated Sands." *Journal of the ASCE Geotechnical Engineering Division* 101 (GT6):551-569.

Dewa, Herose, Saito, and Ohira. 1980. "Earthquake Observations and Analysis of LNG Tank on Pile Foundations." In *Proceedings of the 7th World Conference on Earthquake Engineering,* Istanbul.

Earthquake Engineering Research Institute. 1986. "Geotechnical Engineering." In *Reducing Earthquake Hazards: Lessons Learned from Earthquakes*, Publication 86-02, pp. 23-52. El Cerrito, California: Earthquake Engineering Research Institute.

Gerwick, B. C. Jr. 1982. "Seismic Design of Prestressed Concrete Piles." In *Proceedings of the 9th Congress of the Federation Internationale de la Preconstrainte*, Vol. 2, pp. 60-69.

Gerwick, B. C., Jr., and H. A. Brauner. 1970. *Design of High-Performance Prestressed Concrete Piles for Dynamic Loading.* New York: American Society for Testing and Materials.

Hardman, Scott L., and T. Leslie Youd. 1987. *State of the Art for Assessing Earthquake Hazards in the United States,* Miscellaneous Paper S-73-1. Vicksburg, Mississippi: U.S. Army Waterways Experiment Station.

Hart, E. W. 1988. *Fault-Rupture Hazard Zones in California*, Special Publication 42. Sacramento: California Division of Mines and Geology.

Idriss, I. M. 1989. "Response of Soft Soil Sites During Earthquakes." In *Proceedings of the H. Bolton Seed Memorial Symposium*, Vol. 2, pp. 273-289.

Idriss, I. M. 1985 "Evaluating Seismic Risk in Engineering Practice." In *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering*, Vol 1, p. 155.

Ishihara, Kenji. 1985. "Stability of Natural Deposits During Earthquakes." In *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering*, San Francisco, Vol. 1, pp. 321-376.

Kishida, Hanazuto, and Nakai. 1980. "Damage of Reinforced Precast Concrete Piles During the Miyagi-Oki Earthquake of June 12, 1981." In *Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul*, Vol. 9, p. 461.

Kishida. "Damage of Reinforced Concrete Buildings in Niigata City with Special Reference to Foundation Engineering." *Soils and Foundations* 6(1):71-86.

Koester, J. P., and A. G. Franklin. 1985. *Current Methodologies for Assessing the Potential for Earthquake Induced Liquefaction in Soils,* Report NUREG/CR-4430 RA. Washington, D.C.: U.S. Nuclear Regulatory Commission.

Ledbetter, R. H. 1985. *Improvement of Liquefiable Foundation Conditions Beneath Existing Structures,* Technical Report REMR-GT-2. Vicksburg, Mississippi: U.S. Army Waterways Experiment Station.

Makdisi, F. T., and H. B. Seed. 1978. "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations." *Journal of Geotechnical Engineering* ASCE 104(GT7):849-867.

Margason, E., and M. Holloway. 1977. "Pile Bending During Earthquakes." In *Proceedings of the 6th World Conference on Earthquake Engineering, New Dehli.*

National Research Council, Committee on Earthquake Engineering. 1985. *Liquefaction of Soils During Earthquakes.* Washington, D.C.: National Academy Press.

Nodar, Setsuo, and Satoshi Hayashi. 1980. "Damage to Port Structures in the 1978 Miyagi-Kenoki Earthquake." In *Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul,* Vol. 9, p. 415.

Park, R., and T. J. Falconer. 1983. "Ductility of Prestressed Concrete Piles Subjected to Seismic Loading." *PCI Journal* (September/October).

Poulos, S. J., G. Castro, and J. W. France. 1985. "Liquefaction Evaluation Procedures." *ASCE Journal of Geotechnical Engineering* 111(6).

Rollins, Kyle M. 1987. The Influence of Buildings on Potential Liquefaction Damage. Ph.D. dissertation, University of California, Berkeley.

Seed, H. B. 1987. "Design Problems in Soil Liquefaction." *ASCE Journal of Geotechnical Engineering* 113(8).

Seed, H. B., I. Arango, and C. K. Chan. 1975. *Evaluation of Soil Liquefaction Potential During Earthquakes,* Report EERC 75-28. Berkeley, California: Earthquake Engineering Research Center.

Seed, H. B., and W. H. Peacock. 1971. "Test Procedures for Measuring Soil Liquefaction Characteristics." *Journal of the ASCE Soil Mechanics and Foundation Division* 97(SM8), Paper 8330.

Seed, H. B., and I. M. Idriss. 1971. "Simplified Procedure for Evaluating Soil Liquefaction Potential." *Journal of the ASCE Soil Mechanics and Foundations Division* 97(SM9):1249-1273.

Seed, H. B., Mori, Kenji, and C. K. Chan. 1977. "Influence of Seismic History on the Liquefaction of Sands." *Journal of the ASCE Geotechnical Engineering Division* (April).

Seed, H. B. 1976. "Evaluation of Soil Liquefaction Effects of Level Ground During Earthquakes." In *Liquefaction Problems in Geotechnical Engineering, Proceedings of the ASCE National Convention.*

Seed, H. B., K. Tokimatsu, L. F. Harder, and R. M. Chung. 1985. *The Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations*, Report UBC/EERC-84/15. Berkeley, California: Earthquake Engineering Research Center.

Seed, H. B., I. M. Idriss, and I. Arango. 1983. "Evaluation of Liquefaction Potential Using Field Performance Data." *ASCE Journal of Geotechnical Engineering* 109(3):458-482.

Seed, H. B., and I. M. Idriss. 1982. *Ground Motions and Soil Liquefaction During Earthquakes*, Vol. 5 of *Engineering Monographs on Earthquake Criteria, Structural Design, and Strong Motion Records.* El Cerrito, California: Earthquake Engineering Research Institute.

Seed, R. B., and L. F. Harder. 1989. "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength." In *Proceedings of the H. Bolton Seed Memorial Symposium*, Vol. 2, pp. 351-376.

Shepard, D. A. 1983. "Seismic Design of Concrete Piling." *PCI Journal* (March/April).

Slemmons, D. B., P. Bodin, and X. Zhang. 1989. "Determination of Earthquake Size from Surface Faulting Events." In *Proceedings of the International Seminar on Seismic Zonation*, Guangahou, China, pp. 157-169.

Sugimura. 1981. "Earthquake Damage and Design Method of Piles." In *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm.*

Sugimura. 1980. "Participation Factor of Horizontal Force Applied to Pile Foundation." In *Proceedings of the 7th World Conference on Earthquake Engineering, Istanbul*, Vol. 3, p. 443.

Tamura et al. 1973. "Damage of Bridges During the Tokachi-Oki Earthquake (1968)." In *Proceedings of the Japan Earthquake Engineering Symposium*, pp. 147-153.

Tokimatsu, K., and H. B. Seed. 1987. "Evaluation of Settlements in Sands Due to Earthquake Shaking." *Journal of Geotechnical Engineering* ASCE 113(8):861-878.

Youd, T. L. 1989. "Ground Failure Damage to Buildings During Earthquakes." In *Foundation Engineering--Current Principles and Practices*, Vol. 1, pp. 758-770. New York: American Society of Civil Engineers.

Youd, T. L., and D. M. Perkins. 1987. "Mapping of Liquefaction Severity Index." *Journal of Geotechnical Engineering* ASCE 113(11):1374-1392.

Chapter 5 Commentary

STEEL STRUCTURE DESIGN REQUIREMENTS

5.1 REFERENCE DOCUMENTS: The reference documents presented in this section are the current specifications for the design of steel members, systems, and components in buildings as approved by the American Institute of Steel Construction (AISC), the American Iron and Steel Institute (AISI), the American Society of Civil Engineers (ASCE) and the Steel Joist Institute (SJI).

5.2 STRUCTURAL STEEL SEISMIC REQUIREMENTS: The new *Seismic Provisions for Structural Steel Buildings*, AISC, June 1992 (Ref. 5-3) is adopted by reference and contains, as its name implies, the seismic requirements for structural steel. These AISC provisions are, to the extent possible, based on the terminology and loads in the 1993 Edition of *Minimum Design Loads for Buildings and Other Structures* (ASCE 7) which, in turn, is based on the 1991 *NEHRP Recommended provisions*. The *Seismic Provisions for Structural Steel Buildings*, AISC, 1992 (Ref. 5-3) is adopted by reference by ASCE 7-93. Ref. 5-3 is divided into two parts: Part I is based on the AISC *Load and Resistance Factor Design Specification* (LRFD), Ref. 5-1. Part II is based on the AISC *Allowable Stress Design Specification* (ASD), Ref. 5-2, but the allowable stresses are multiplied by 1.7 to convert to nominal strengths. Simplified ϕ factors are given in Part II to allow the user to develop design strengths that can be used with the limit state seismic loads of the *NEHRP Recommended Provisions*. The reader is referred to Ref. 5-3 for an excellent commentary on the provisions for structural steel.

Based on the unexpected structural damage to steel moment frames from the 1994 Northridge earthquake and pending the results of ongoing and future research studies, use of the welded flange and welded or bolted web special moment frame connection to column connection in Sec. 8.2.c of Ref. 5-3 should be supplemented by the addition of steel flange plates or equivalent measures. Accordingly, the prescriptive Sec. 8.2.c has been suspended and replaced with a performance requirement intended to achieve ductile yielding of the special moment frame under severe cyclic loads. The expected strength ratios of the beam and column materials should be considered. Good welding practice must be followed including all the requirements of ANSI/AWS D1.1-94 such as qualified welders (5.3), preheat (4.2), technique (Sec. 4, Parts B, C and D), inspection (Sec. 6 and 8, Part D), and the possible use of electrodes capable of depositing notch-tough weld metal such as found in ANSI/AWS A5.1, 5.5, 5.20, and 5.29.

5.2.1 REQUIREMENTS FOR SPECIAL CONCENTRICALLY BRACED FRAMES (SCBF): Special concentrically braced frames (SCBF) are those concentrically braced frames which are specially designed to ensure stable and ductile behavior in the event of a major earthquake. The values of R and C_d factors are selected such that they are consistent with those currently specified for framing systems which depict similar ductile behavior. Studies (Goel, 1992b; Hassan and Goel, 1991; Tang and Goel, 1987) have shown that special concentrically braced frames designed by recommended provisions possess excellent ductility and energy dissipation capacity (Figures C5.2.1a and C5.2.1b) and, consequently, their response can be significantly superior to SMF when subjected to the same ground motions.

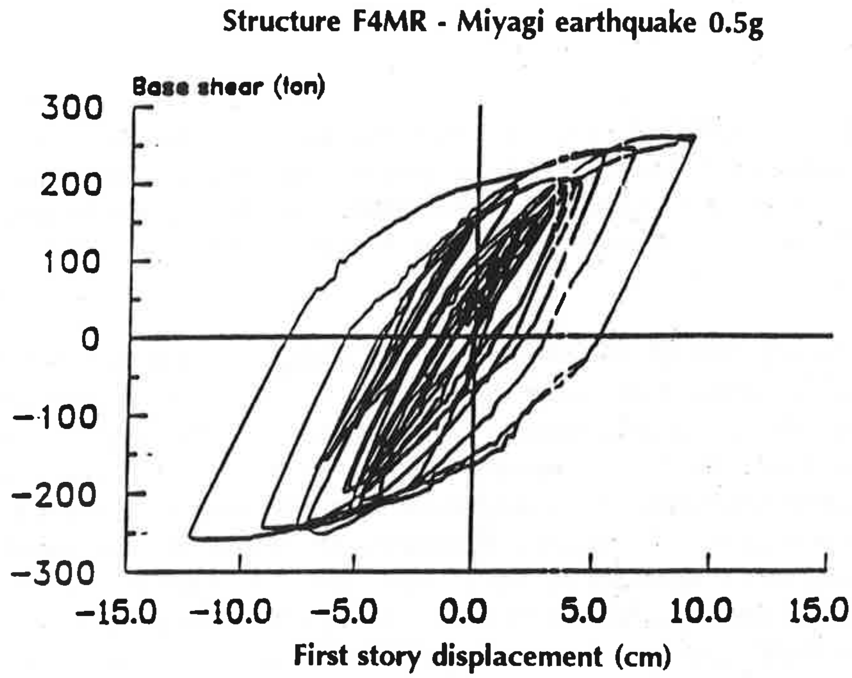


FIGURE C5.2.1a Story hysteretic loops of dual SCBF.

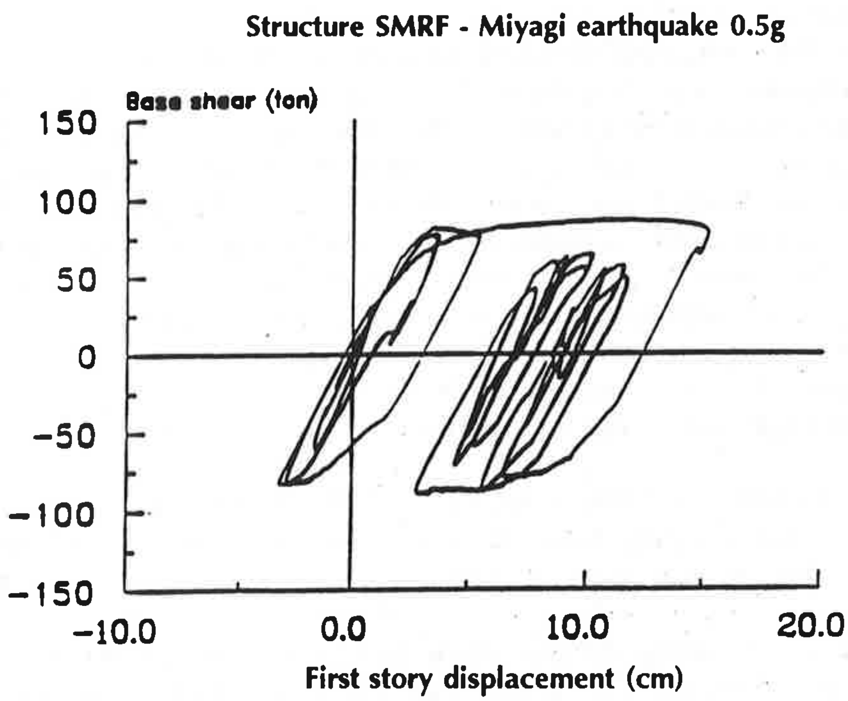


FIGURE C5.2.1b Story hysteretic loops of a SMRF.

Sec. 9.2.a of Ref. 5-3: Slenderness limit of $720/\sqrt{F_y}$ is not necessary when the bracing members are detailed for ductile behavior. In fact, studies have shown that post-buckling cyclic fracture life of bracing members generally decreases with the increase in slenderness ratio (Goel and Lee, 1992; Tang and Goel, 1989).

Sec. 9.2.b of Ref. 5-3: Brace strength reduction factor of 0.8 used for (ordinary) CBF has little influence on inelastic seismic response of SCBF when the ductility of braces is ensured.

Sec. 9.2.d of Ref. 5-3: Width-thickness ratios more restrictive than those used for (ordinary) CBF are necessary in order to prevent premature fracture of bracing members due to local buckling under cyclic post-buckling deformation.

Sec. 9.2.e of Ref. 5-3: Stitch requirements for built-up bracing members are added as they appeared in the 1991 *Provisions*. These provisions are more appropriate for ductile behavior of built-up bracing members than those in Ref. 5-1.

Sec. 9.4.a of Ref. 5-3: A beam intersected by V-bracing must be capable of resisting the effects of unbalanced forces due to brace buckling and yielding, in combination with appropriate gravity loads. This is necessary to prevent undesirable deterioration of lateral strength of the frame. Similarly, adequate lateral support at brace-to-beam intersection is necessary in order to prevent adverse effects of possible lateral-torsional buckling of the beam.

Sec. 9.4.b of Ref. 5-3: It is not desirable to permit use of K-bracing in SCBF.

Sec. 9.5 of Ref. 5-3: Requirements for SCBF should not be waived for low buildings, since somewhat lower design forces are proposed for SCBF than those specified for (ordinary) CBF. These buildings can be designed by (ordinary) CBF provisions.

Sec. 9.5 Columns (New) of Ref. 5-3: In the event of a major earthquake, columns in concentrically braced frames can undergo significant bending beyond the elastic range after buckling and yielding of the braces. Therefore, columns for use in SCBF must have adequate compactness, and shear and moment capacities (including those at the splices) in order to maintain their lateral strength during cyclic deformation of the frame. Studies have shown that buckling and yielding of bracing members in concentrically braced frames can occur at story drift levels as low as 1/3 percent (Goel, 1992a; Tang and Goel, 1987).

5.3 COLD-FORMED STEEL SEISMIC REQUIREMENTS: The allowable stress and allowable load levels in Ref. 5-4 are incompatible with the force levels in Chapter 2 of the *Provisions*. It is therefore necessary to modify the provisions of Ref. 5-4 for use with the *Provisions*. Ref. 5-5 and 5-6 are both based on LRFD and thus are consistent with the force levels in Chapter 2 of the *Provisions*. As such, only minor modifications are needed to correlate those load factors for seismic loads to be consistent with these provisions. The modifications of all of the reference documents affect only designs involving seismic loads.

5.4 SEISMIC REQUIREMENTS FOR STEEL DECK DIAPHRAGMS: Since the design values for steel deck are based on allowable loads, it is necessary to present a method of deriving design strengths. Two ϕ values are presented--0.60 for steel deck that is mechanically attached and 0.50 for welded steel deck. These factors are consistent with current proposals being circulated for inclusion in updates of Ref. 5-5.

5.5 STEEL CABLES: The provisions of Sec. 5.5 are virtually unchanged from previsous editions. Although the provisions in Ref. 5-8 are dated, they are the only ones available and there was no sentiment to eliminate them from the *Provisions*. The allowable stress levels of steel cable structures specified in Ref. 5.8 are modified for seismic load effects. The value of $1.5T_4$ was chosen as a reasonable value to compare with increases given to other working stress levels.

5.6 SEISMIC PROVISIONS FOR STEEL STRUCTURAL MEMBERS: The provisions of this section direct the user to use any of the reference documents for seismic design of steel members in areas of low and moderate seismic hazard. In Seismic Performance Category D and E buildings, the user is required to use the additional provisions of Ref. 5-3 for structural steel buildings and Sec. 5.7 for light-framed walls.

5.7 LIGHT-FRAMED WALLS: The provisions of this section apply to buildings framed with cold-formed steel studs and joists. Lateral resistance is typically provided by diagonal braced (braced frames) or wall sheathing material. This section is only required for use in Seismic Performance Category D and E. The required strength of connections is intended to assure that inelastic behavior will occur in the connected members prior to connection failure. Since pull-out of screws is a sudden or brittle type of failure, designs using pull-out to resist seismic loads are not permitted. Where diagonal members are used to resist lateral forces, the resulting uplift forces must be resolved into the foundation or other frame members without relying on the bending resistance of the track web. This often is accomplished by directly attaching the end stud(s) to the foundation, frame, or other anchorage device.

REFERENCES

Goel, S. C. 1992a. "Cyclic Post Buckling Behavior of Steel Bracing Members." In *Stability and Ductility of Steel Structures under Cyclic Loading*, pp. 75-04. CRC Press.

Goel, S. C. 1992b. "Earthquake Resistant Design of Ductile Braced Steel Structures." In *Stability and Ductility of Steel Structures under Cyclic Loading*, pp. 297-308. CRC Press.

Goel, S. C., and S. Lee. 1992. "A Fracture Criterion for Concrete-Filled Tubular Bracing Members under Cyclic Loading." In *Proceedings*, ASCE Structures Congress, San Antonio, Texas, April 13-15, pp. 922-925.

Goel, S. C., and A. Itani. N.d. "Seismic Behavior of Open Web Truss Moment Frames", Paper under review for publication in *Journal of Structural Engineering*, ASCE.

Goel, S. C., and A. Itani. N.d. "Seismic Resistant Special Truss Moment Frames", Paper under review for publication in *Journal of Structural Engineering*, ASCE.

Goel, S. C, and H. S. Basha. N.d. "Special Truss Moment Frames with Vierendeel Segment", Paper under review for publication in *Journal of Structural Engineering*, ASCE.

Hassan, O., and S. C. Goel. 1991. *Seismic Behavior and Design of Concentrically Braced Steel Structures*, Report UMCE 91-1. Ann Arbor: Department of Civil Engineering, The University of Michigan.

Itani, A., and S. C. Goel. 1991. *Earthquake Resistant Design of Open Web Framing Systems*, Research Report UMCE 91-21. Ann Arbor: Department of Civil Engineering, The University of Michigan.

Klippstein, K., and T. Tarpy. 1992. *Shear Resistance of Walls with Steel Studs*, Report CF 92-2. American Iron and Steel Institute.

Tang, X., and S. C. Goel. 1989. "Brace Fractures and Analysis of Phase I Structure," *Journal of Structural Engineering*, ASCE 115(8):1960-1976.

Tang X., and S. C. Goel. 1987. *Seismic Analysis and Design Considerations of Braced Steel Structures*, Report UMCE 87-4. Ann Arbor: Department of Civil Engineering, The University of Michigan.

Chapter 6 Commentary

CONCRETE STRUCTURE DESIGN REQUIREMENTS

6.1 REFERENCE DOCUMENT: The main concern of Chapter 6 is the proper detailing of reinforced concrete construction for earthquake resistance. The bulk of the detailing requirements in this chapter are contained in Ref. 6-1, *Building Code Requirements for Reinforced Concrete*, ACI 318-89 (Revised 1992). The commentary for ACI 318-89 (Revised 1992) contains a valuable discussion of the rationale behind detailing requirements that is not repeated here.

6.1.1 MODIFICATIONS TO REF. 6-1: The modifications noted for ACI 318-89 (Revised 1992) are: changes in load factors necessary to coordinate with the equivalent yield basis of this document; additional definitions necessary for seismic design requirements for structural systems composed of precast elements; and changes that incorporate certain features of the detailing requirements for reinforced concrete that have been adopted into the 1991 *Uniform Building Code*.

Included as Sec. 6.1.1.4 is a statement on reinforced concrete structural systems incorporating precast concrete elements. One design alternative is emulation of monolithic reinforced concrete construction. The other alternative is the use of the unique properties of precast elements interconnected predominately by dry joints. For the first alternative Sec. 6.1.1.5, 6.1.1.7 and 6.1.1.8 define design procedures ensuring that the resulting structural systems have strength and stiffness characteristics equivalent to those for monolithic reinforced concrete construction. The existing code requirements for monolithic construction then apply for all but the connections. The second alternative, the Appendix to Chapter 6, is included for information and for trial design by users.

Procedures for structural system composed from precast elements interconnected predominately by dry joints are included as an appendix because the existing state of knowledge makes it premature to propose code provisions based on that information. The complexity of structural systems, configurations and details possible with precast concrete elements requires:

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.

Research conducted to date (Cheok and Lew, 1991; Elliott et al., 1992; Englekirk, 1987; French et al., 1989; BSSC, 1987; Hawkins and Englekirk, 1987; Jayashanker and French, 1988; Mast, 1992; Nakaki and Englekirk, 1991; Neille, 1977; New Zealand Society, 1991; Pekau and Hum, 1991; Powell et al., 1993; Priestley and Tao, 1991; Stanton et al., 1986; Stanton et al.,

1991) documents concepts for design using dry connections and the behavior of structural systems and subassemblages composed of precast elements both at and beyond peak strength levels for nonlinear reversed cyclic loadings, and provides the basis for the appendix.

Emulation of Monolithic Construction Using Strong Connections: For emulation of the behavior of monolithic reinforced concrete construction, Sec. 6.1.1.5 provides two alternatives. Sec. 21.2.2.6 in Sec. 6.1.1.5 covers structural systems with "strong" connections. Sec. 21.2.2.7 in Sec. 6.1.1.5 covers structural systems with "ductile" connections.

For frame systems that use strong connections, Sec. 21.2.2.6 and 21.2.7, the different connection categories envisaged are shown in Figure C6.1.1-1. Considerable freedom is given to locating the nonlinear action zones (plastic hinges) along the length of the precast member. However, those hinges must be separated from the connection by a distance of at least three quarters of the member's depth. Wet-joint connections are permitted at the strong connection but not at the hinge location.

Provision 21.2.7.2 makes the strength required for a strong connection dependent on the distances hinges are separated from that connection, the strengths of those hinges and the nonlinear deformation mechanism envisaged. The conditions described by Sec. 21.2.7.2 for a beam to continuous column connection are shown in Figure C6.1.1-2, which is an adaptation of Figure R21.3.4 of Ref. 6-1. Because the strong connection must not yield or slip; its nominal strengths, $S_n^{CONNECTION}$, in both flexure and shear must be greater than those corresponding to development of the probable strengths M_{pr1} and M_{pr2} at the hinge locations. Figure C6.1.1-2b, illustrates the situation for flexure. Per Ref. 6-1 moments M_{pr1} and M_{pr2} are determined using a strength reduction factor of 1.0 and reinforcing steel stresses of at least $1.25f_y$.

For columns above the ground floor, moments at a joint may be limited by flexural strengths of the beams framing into that joint. However, for a strong column-weak beam deformation mechanism, dynamic inelastic analysis and studies of strong motion measurements have shown that beam end moments are not equally divided between top and bottom columns even where those columns have equal stiffness. Elastic analysis predicts moments as shown in Figure C6.1.1-3a while the actual situation is likely to be as shown in Figure C6.1.1-3b. Accordingly, provision 21.2.7.3 is included for the midheight column connection.

Emulation of Monolithic Construction Using Ductile Connections: In Sec. 6.1.1.5, provision 21.2.2.7 covers the situation for both frame and panel systems where the connections used have adequate nonlinear response characteristics and it is not necessary to ensure plastic hinges remote form the connections. Usually physical testing is required to prove that a connection has the necessary nonlinear response characteristics. Warnes (1992) and Yee (1991) have documented one connection type that has such characteristics. Minimum requirements for proportioning connections meeting Sec. 21.2.2.7 are specified in Sec. 6.1.1.8. Those requirements are necessary but not sufficient conditions to satisfy Sec. 6.1.1.8.

The requirement of Sec. 21.2.8.1 is intended to make the designer consider the likely deformations of any proposed precast structure vis-a-vis those of the same structure composed of monolithic reinforced concrete before claiming that the precast form emulates monolithic construction. For example, a designer might propose a shear wall composed of multiple precast panels over its length and height that are connected vertically but not horizontally. With ductile vertical connections, that precast wall could be made to meet all the requirements of Sec. 21.2.8 except Sec. 21.2.8.1. That wall could therefore not be designed using this provision.

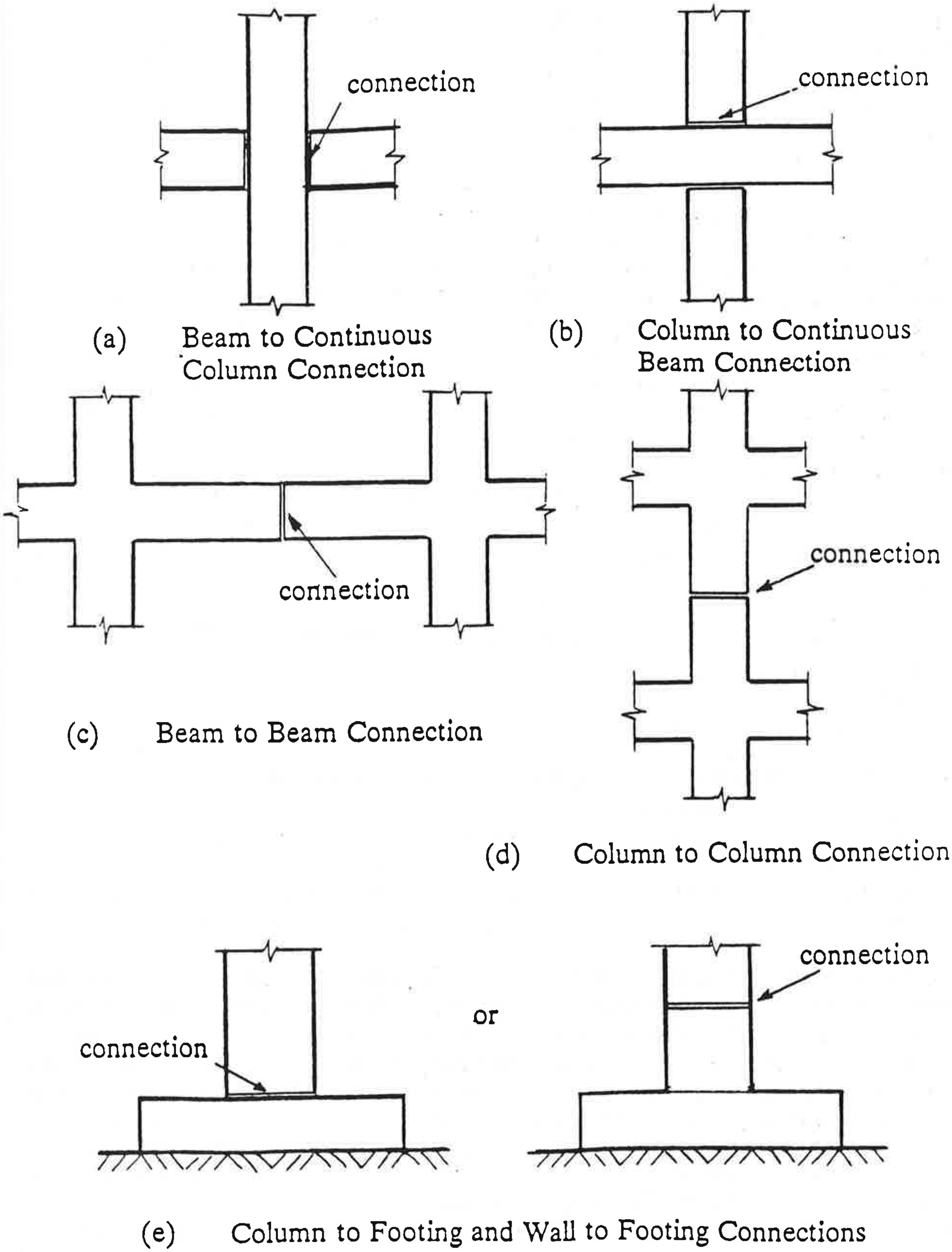


FIGURE C6.1.1-1 Connection categories.

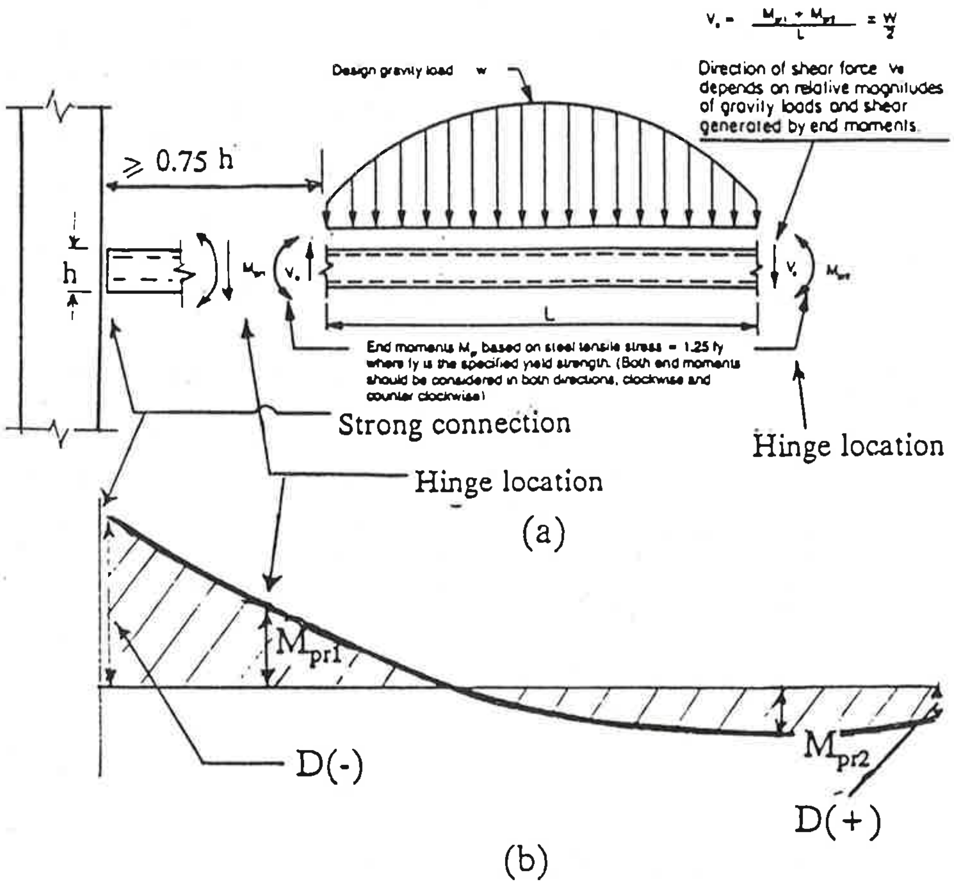


FIGURE C6.1.1-2 Design forces for strong connections between beams and continuous columns.

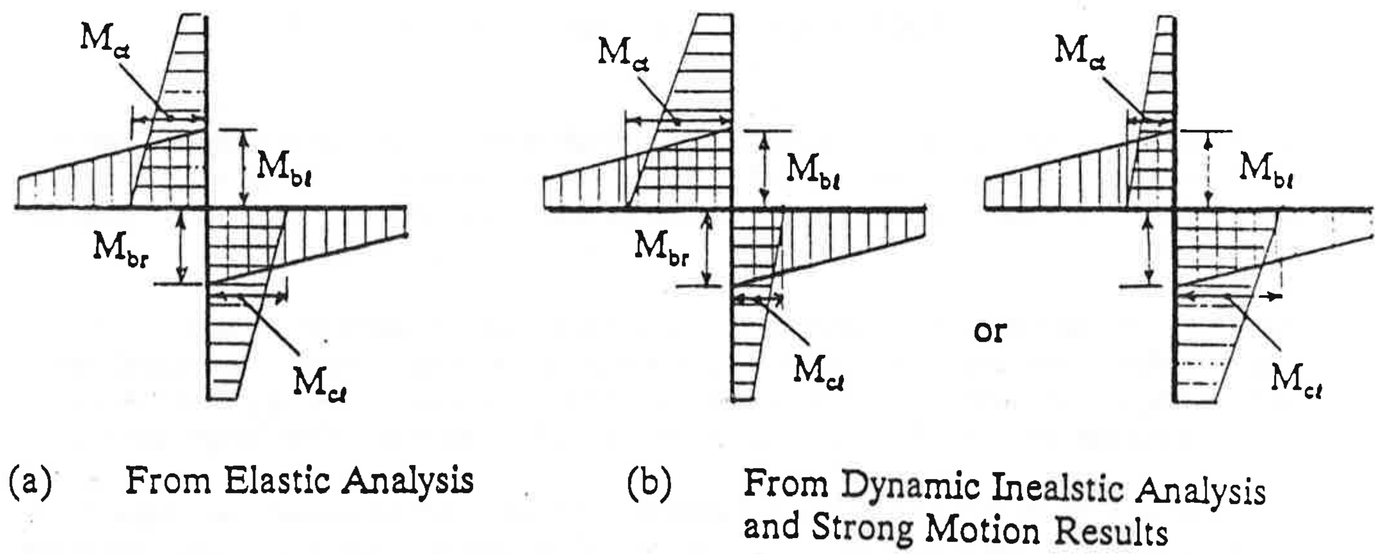
The requirement of Sec. 21.2.8.2 can usually be achieved only with the use of special mechanical connectors, such as the sleeve splice of Yee (1991) or welding of the reinforcing bars.

Sec. 21.2.8.4 recognizes that if the monolithic wall of Figure C6.1.1-4a, Part a, is composed of precast elements, as shown in Figure C6.1.1-4b, then the shear force acting on the connection at A-A can be limited by the shear capacity of the precast element above A-A, by the shear for slip along the connection, or by the probable connection moment capacity, M_{pr}. That moment corresponds to the value of H that causes a stress of $1.25f_y$ in the boundary reinforcement continuous across A-A. When the moment due to H causes a stress of $1.25f_y$ in the boundary reinforcement, the shear causing slip along the connection is less than if the steel stress was less than $1.25f_y$. The shear to cause slip decreases as the crack width increases. Only when the steel stress is limited to f_y can the shear strength be taken as that calculated by Sec. 6.7. The probable shear strength is taken as that documented by Mueller (1989) and Wood (1990) for precast and monolithic shear walls, respectively.

Provision 21.2.8.8 recognizes that, as documented by Mattock (1974 and 1977), the shear friction capacity decreases as the degree of shear reversal increases and, for fully reversed cyclic loading is 20 percent less than that for monotonic loading.

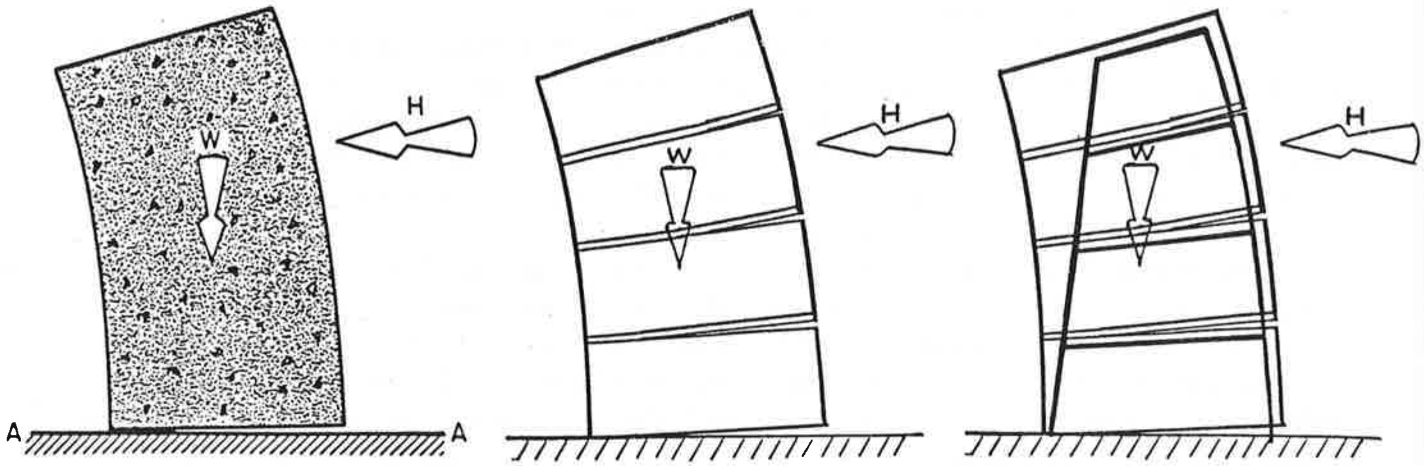
The shear carrying mechanism of the monolithic wall of Figure C6.1.1-4a and that of the precast wall of Figure C6.1.1-4b are distinctly different when the overturning moment causes yielding of the boundary reinforcement and therefore opening of the horizontal connections. Lateral shears can then be transferred through compressed concrete only and the precast wall must be provided at the upper edge of the panel sufficient to balance the horizontal component of the force in the compression diagonal.

Use of Prestressing Tendons: Sec. 6.1.1.6 defines conditions under which prestressing tendons can be used, in conjunction with deformed reinforcing bars, in frames resisting earthquake forces. As documented by Ishizuka and Hawkins (1987), if those conditions are met no modification is necessary to the R and C_d factors of Table 2.2.2 when prestressing is used. Satisfactory seismic performance can be obtained when prestressing amounts greater than those permitted by Sec. 6.1.1.6 are used. For example, the connection of Figure C6A-3 has a satisfactory performance for a probable strength of 180 kips and a reversed deformation of 10 mm. However, as documented by Park and Thompson (1977) and Thompson and Park (1980) and required by the combination of New Zealand Standards 3101:1982 and 4203:1992, ensuring that satisfactory performance requires modification of the R and C_d factors.



In all cases $M_{bl} + M_{br} = M_{cl} + M_{ct}$

FIGURE C6.1.1-3 Moments at beam-to-column connections.



(a) Monolithic Wall (b) Precast Wall (c) Precast Wall

Loading Conditions Load Resisting Mechanism

FIGURE C6.1.1-4 Conditions for walls.

6.1.1.12: The minimum thicknesses for concrete diaphragms reflect current usage in joist and waffle systems and topping slabs for precast floor and roof systems. Bonding of the top slab provides restraint against slab buckling which, for an untopped joist or waffle system, is provided by the webs.

6.1.1.13 Coupling Beams: Short-span coupling beams between shear walls, under reversing loads simulating earthquakes, have been experimentally investigated by many researchers (Barney et al., 1978; Bertero and Popov, 1975; Brown and Jirsa, 1971; Hirosawa et al., 1973; Ma et al., 1976; Paulay and Binney, 1974; Scribner and Wight, 1978; Shiu et al., 1978; Wight and Sozen, 1975).

The tests indicate that short flexural members with small clear-span-to-effective-depth ratios behave differently than slender flexural members. When short coupling beam specimens were subjected to inelastic load cycles, large flexural cracks formed at both beam ends. With increasing load reversals, cracks from each direction of loading interconnected, forming a vertical plane of weakness at each end of a beam. Thus, instead of a conventional truss mechanism, shear transfer across the plane of weakness was provided primarily by aggregate interlock and shear friction. Under subsequent inelastic load cycles, the shear resisting mechanism deteriorated rapidly resulting in a loss of load capacity by "sliding shear."

It was found (Barney et al. , 1978; Paulay and Binney, 1974; Shiu et al., 1978) that increasing the number of hoop stirrups was not effective in improving resistance against sliding shear. Therefore, various configurations of special shear reinforcement to improve seismic performance of short coupling beams were tested (Barney et al., 1978; Bertero and Popov, 1974; Paulay and Binney, 1974; Scribner and Wight, 1978). Full-length diagonal reinforcement was found to produce the greatest energy dissipation and deformation capacities.

In tests at the Portland Cement Association (Shiu et al., 1978), beam specimens with clear-span-to-effective-depth ratio of 2.8 were able to sustain over 34 reversing load cycles. A maximum nominal shear stress of $12.5\sqrt{f_c'}$ was recorded and a maximum imposed deformation of over nine times yield deflection was measured. In addition, the specimen was able to dissipate three times more energy than a comparable specimen without diagonal shear reinforcement. No sign of sliding shear was observed at completion of testing. Similar results have been reported by Paulay and Binney (1974). In both investigations, diagonal shear reinforcement was designed by the proposed equation to resist the total shear force by truss action.

Based on test results, Paulay (1977) recommended that diagonal shear reinforcement be used to carry 75 percent of the induced shear in flexural members when nominal design shear stress under load reversals is larger than $3\sqrt{f_c'}$ psi. When nominal shear stress exceeds $4.5\sqrt{f_c'}$ psi, he recommended that diagonal reinforcement should carry 100 percent of the induced cyclic shear forces.

For clear-span-to-effective-depth ratios greater than 4.0, PCA tests (Barney et al., 1978; Shiu et al., 1978) indicated that specimens with conventional longitudinal reinforcement resisted over 46 inelastic reversing load cycles. At the same time, maximum deformation of 13 times yield deflection was measured. Therefore, experimental data have shown that beams with clear-span-to-effective-depth ratios greater than 4.0 do not require diagonal reinforcement. In addition, diagonal reinforcement is not very effective in beams with clear-span-to-effective-depth ratios greater than 4.0.

It is permitted to use reinforcement arrangements without the diagonal bars if it is assumed that the coupling beams may fail early in the design earthquake and the consequences of such failures are taken into account. The consequences include, but are not necessarily limited to, reductions in stiffness due to lack of coupling between walls as well as debris or deformed members blocking exits and damage or failure of nonstructural components and cladding and the connections thereto due to increased drifts. The vertical load carrying capacity of the structure must not be impaired.

Even if the coupling beams are expected to fail early in the earthquake, the full design strength of the coupling beams still must be provided if they are assumed to be part of the seismic force resisting system. This is to reduce the likelihood that the transition of shear wall behavior from coupled to uncoupled action will begin at unduly low seismic loads.

6.2 BOLTS AND HEADED STUD ANCHORS IN CONCRETE: The allowable loads on anchor bolts have been chosen to suit the capacity reduction factors in this document.

6.2.2 BOLTS AND HEADED STUD ANCHORS: These provisions follow those given in the *Uniform Building Code* modified by recent improvements in shear capacity calculation given in the *PCI Design Handbook*, Fourth Edition.

While the provisions do not prohibit the use of single anchor connections, it is considered necessary to use at least two anchors in any load-carrying device whose failure might lead to collapse.

The provisions generally relate to groups of anchors attaching a loaded steel plate to the concrete surface. The thickness of this connector plate also is a design consideration and must be adequate to allow the anchors to perform in group action if the calculated design strengths are to be realized. A plate thickness of not less than one half the diameter of the anchor shank is recommended.

These provisions are intended to provide proper strength of connections. To achieve adequate ductility for seismic or other dynamic loads, use of auxiliary reinforcement for confining concrete or for direct load transfer should be considered. A number of tests have shown that hairpins or similar reinforcement confining the concrete engaged by the anchors and running through the failure surface into the adjacent concrete provides enhancement of a connection's ductility under dynamic loading. No clear recommendations as to the design of such reinforcement have been suggested, but its use is highly recommended in all anchor connections and particularly those subject to seismic or other dynamic loading.

Tests have shown that there are consistent shear ductility variations between bolts anchored to drilled or punched plates with nuts and connections using welded, headed studs. Recommendations for design are not presently available, but this should also be considered in critical connections subject to dynamic or seismic loading.

6.3 CLASSIFICATION OF MOMENT FRAMES:

6.3.1 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. 6-1.

6.3.2 INTERMEDIATE MOMENT FRAMES AND 6.3.3 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. 6-1. Two sets of moment frame detailing requirements are defined in Ref. 6-1, 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. 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. 6-1 for a fuller discussion of the requirement for the special frame):

1. 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.

2. 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.

3. 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.

4. 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.

5. 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.

6. 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.

7. 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.

8. Locations for splices of reinforcement are more tightly controlled for the special frame.

9. 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.

6.4 SEISMIC PERFORMANCE CATEGORY A: Construction qualifying under Category A may be built with no special detailing requirements for earthquake resistance. Special details for ductility and toughness are not required in Category A.

6.5 SEISMIC PERFORMANCE CATEGORY B: Special details for ductility and toughness are not required in Category B.

6.6 SEISMIC PERFORMANCE CATEGORY C: A frame used as part of the lateral force resisting system in Category C as identified in Table 2.2.2 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. 6-1 except the provisions of Sec. 21.6 of Ref. 6-1 do not apply.

6.7 SEISMIC PERFORMANCE CATEGORIES D AND E: The requirements conform to current practice in the areas of highest seismic hazard.

REFERENCES

Precast/Prestressed Concrete

ACI Committee 550. 1993. "Design Recommendations for Precast Concrete Structures," *ACI Structural Journal* 90 (1):115-121.

Applied Technology Council. 1981. *Proceedings of a Workshop on Design of Prefabricated Concrete Buildings for Earthquake Loads*, Report ATC-8.

Building Seismic Safety Council. 1987. *Guide to Use of the NEHRP Recommended Provisions in Earthquake Resistant Design of Buildings*, 1985 Edition. Washington, D.C.: FEMA.

Cheok, G. S., and H. S. Lew. 1991. "Performance of Precast Concrete Beam-to-Column Connections Subject to Cyclic Loading," *PCI Journal* 36 (3):56-67.

Clough, D. P. 1986. "A Seismic Design Methodology for Medium-Rise Precast Concrete Buildings." In *Proceedings of Seminar on Precast Construction in Seismic Zones*, JSPS/NSF, Tokyo, October.

Englekirk, R. E. 1987. "Concepts for the Development of Earthquake Resistant Ductile Frames of Precast Concrete," *PCI Journal* 32(1).

Elliott, K. S., G. Davies, and W. Omar. 1992. "Experimental and Theoretical Investigation of Precast Concrete Hollow-Cored Slabs Uses as Horizontal Floor Diaphragms," *The Structural Engineer* 70(10):175-187.

French, C. W., M. Hafner, and V. Jayashanker. 1989. "Connections Between Precast Elements Failure Within Connection Region," *ASCE Journal of Structural Engineering* 115(12):3171-3192.

Hawkins, N. M., and R. E. Englekirk. 1987. "U.S.-Japan Seminar on P/C Concrete Construction in Seismic Zones," *PCI Journal* 32(2).

Jayashanker, V., and C. E. French. 1988. *An Interior Moment Resistant Connection Between Precast Elements Subjected to Cyclic Lateral Loads*, Structural Engineering Report 87-10. Mineapolis: University of Minnesota.

Mast, R. F. 1992. "A Precast Concrete Frame System for Seismic Zone Four," *PCI Journal* 37(1):50-64.

Mattock, A. H. 1977. *Shear Transfer Under Cyclically Reversing Loading Across an Interface Between Concretes Cast at Different Times*, Report SM 77-1. Seattle: Department of Civil Engineering, University of Washington.

Mattock, A. H. 1974. *The Shear Transfer Behavior of Cracked Monolithic Concrete Subject to Cyclically Reversing Shear*, Report SM 74-4. Seattle: Department of Civil Engineering, University of Washington.

Menegotto, M. 1994. "Seismic Diaphgram Behavior of Untopped Hollow-Core Floors." In *Proceedings*, Federation International of Prestressing (FIP) Congress, Washington, D.C., May.

Mueller, P. 1989. "Hysteric Behavior of Precast Panel Walls." In *Proceedings*, Seminar on Precast Concrete Construction in Seismic Zones, JSPS/NSF, Tokyo, 1989, Vol. I, pp. 127-142.

Nakaki, S. D., and R. E. Englekirk. 1991. "PRESS Industry Seismic Workshops: Concept Development," *PCI Journal* 36(5):54-61.

Neille, D. S. 1977. Behavior of Headed Stud Connections for Precast Concrete 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.

New Zealand Society of Earthquake Engineering. 1991. *Guidelines for the Use of Structural Precast Concrete in Buildings.*

Pekau, O. A., and D. Hum. 1991. "Seismic Response of Friction-Jointed Precast Panel Shear Walls," *PCI Journal* 36(2):56-71.

Powell, G., F. Filippou, V. Prakash, and S. Campbell. 1993. "Analytical Platform for Precast Structural Systems." In *Proceedings*, ASCE Structures Congress '93. New York: ASCE.

Priestley, M. J. N. 1991. "Overview of PRESS Research Program," *PCI Journal* 36(4):50-57.

Priestley, M. J. N., and J. R. Tao. 1991. "Seismic Response of Precast-Prestressed Frames with Partially Debonded Tendons," Second Meeting of US-Japan Joint Technical Coordinating Committee for PRESS, Tsukuba, Japan, November.

Stanton, J. F., R. G. Anderson, C. W. Dolan, and D. E. McClearly. 1986. *Moment-Resisting and Simple Connections*, Research Report 1/4, Prestressed Concrete Institute.

Warnes, C. E. 1992. "Precast Concrete Connection Details for All Seismic Zones," *Concrete International* 14(11):36-44.

Wood, S. L. 1990. "Shear Strength of Low-Rise Reinforced Concrete Walls," *ACI Structural Journal* 87(1):99-107.

Yee, A. A. 1991. "Design Considerations for Precast Prestressed Concrete Building Structures in Seismic Areas," *PCI Journal* 36(3):40-55.

Prestressed and Partially Prestressed Concrete

ACI-ASCE Committee 423. 1985. "Recommendations for Concrete Members Prestressed with Unbonded Tendons," *ACI Manual for Concrete Practice*, Part 3, p. 423.3-1 - 423.3-R16. Detroit, Michigan: American Concrete Institute.

Hirosawa, M., M. Ozaki, and M. Wakabayashi. 1973. "Experimental Study on Large Models of Reinforced Concrete Columns." In *Proceedings of the Fifth World Conference on Earthquake Engineering, Rome*.

Ishizuka, T., and N. M. Hawkins. 1987. *Effect of Bond Deterioration on the Seismic Response of Reinforced and Partially Prestressed Concrete and Ductile Moment Resistant Frames*, Report SM 87-2. Seattle: University of Washington, Department of Civil Engineering.

JSPS/NSF. 1986. *Proceedings of the Seminar on Precast Construction in Seismic Zones*, Vol. 1.

Ma, S. M., V. V. Bertero, and E. P. Popov. 1976. *Experimental and Analytical Studies on the Hysteretic Behavior of Reinforced Concrete Rectangular and T-Beams*, Report EERC 72-2. Berkeley: University of California.

Park, R., and K. J. Thompson. 1977. "Cyclic Load Tests on Prestressed and Partially Prestressed Beam-Column Joints," *PCI Journal* 22(5):84-110.

Paulay, T. 1977. "Capacity Design of Reinforced Concrete Ductile Frames." In *Proceedings of the Workshop on ERCBC*, University of California.

Paulay, T., and J. R. Binney. 1974. "Diagonally Reinforced Coupling Beams of Shear Walls." In *Shear in Reinforcement Concrete*, Vol. 2, Publication SP-42, pp. 579-598. Detroit, Michigan: American Concrete Institute.

Scribner, C. F., and J. K. Wight. 1978. *Delaying Shear Strength Decay in Reinforced Concrete Flexural Members Under Large Load Reversals*, University of Michigan, Department of Civil Engineering Report UMEE 78R2 to the National Science Foundation.

Shiu, K. N., G. B. Barney, A. E. Fiorato, and W. G. Corley. 1978. "Reversing Load Tests of Reinforced Concrete Coupling Beams." In *Proceedings of the Central American Conference on Earthquake Engineering, San Salvador*.

Thompson, K. J., and R. Park. 1980. "Seismic Response of Partially Prestressed Concrete," *ASCE Journal of the Structural Division* 106(ST8):1755-1775.

Appendix to Chapter 6

REINFORCED CONCRETE STRUCTURAL SYSTEMS COMPOSED OF INTERCONNECTED PRECAST ELEMENTS

PREFACE: The provisions for reinforced concrete structural systems composed of precast elements in the body of the 1994 *Provisions* are for precast systems emulating monolithic reinforced concrete construction. However, one of the principal characteristics of precast systems is that they often are assembled using dry joints where connections are made by bolting, welding, post-tensioning, or other similar means. Research conducted to date documents concepts for design using dry joints and the behavior of subassemblages composed from interconnected precast elements both at and beyond peak strength levels for nonlinear reversed cyclic loadings (Applied Technology Council, 1981; Cheok and Lew, 1992; Clough, 1986; Eliott et al., 1987; Hawkins and Englekirk, 1987; Jayashanker and French, 1988; Mast, 1992; Nakaki and Englekirk, 1991; Neille, 1977; New Zealand Society, 1991; Pekau and Hum, 1991; Powell et al., 1993; Priestley, 1991; Priestley and Tao, 1992; Stanton et al., 1986; Stanton et al., 1991). This appendix is included for information and as a compilation of the current understanding of the performance under seismic loads of structural systems composed from interconnected precast elements. It is considered premature to base code provisions on this resource appendix; however, user review, trial designs, and comment on this appendix are encouraged. Please direct such feedback to the BSSC.

The only design approach currently validated adequately for codification for construction using precast elements is that of emulation of monolithic reinforced concrete construction. Yet, in regions of moderate and low seismicity, it is reasonable to expect that structural systems of adequate strength, stiffness, and energy dissipation can be constructed from precast concrete elements using bolting, welding, or similar means that involve dry connections only. The objective of this appendix is to provide a framework within which such systems can begin to be codified for design purposes.

Tests by Stanton et al. (1986) have demonstrated that many of the moment resisting dry connections typically utilized for precast concrete construction for gravity loadings have adequate behavior for monotonic but not reversed loading. For reversed loadings, such connections lack ductility whenever the connection is made by welding or bolting. For example, as illustrated in Figure C6A-1, the corbel connection shown in (a) functions well for gravity loadings (b) but not for loading reversals (c) through (f). In the latter case, the corbel is an impediment to ductility because the connection is not detailed carefully enough. For negative moment loading, the beam end rotates about the edge of the corbel causing a prying action on the negative moment connection to the column and marked secondary stresses in the reinforcement, inserts, or welds that make up that connection. The prying causes kinking of the reinforcement initiating spalling or splitting of the concrete surrounding that reinforcement. That action, combined with splitting of the beam end or the corbel edge for positive moment loadings, results in the strength rapidly dropping below acceptable levels with load reversals and increasing rotations. However, that does not mean that all corbels *per se* are bad for reversed loadings. Shown in Figure C6A-2 is an inverted T-beam detail that has been developed to provide positive connection to the corbel while minimizing difficulties associated with beam shrinkage and beam loading reversals.

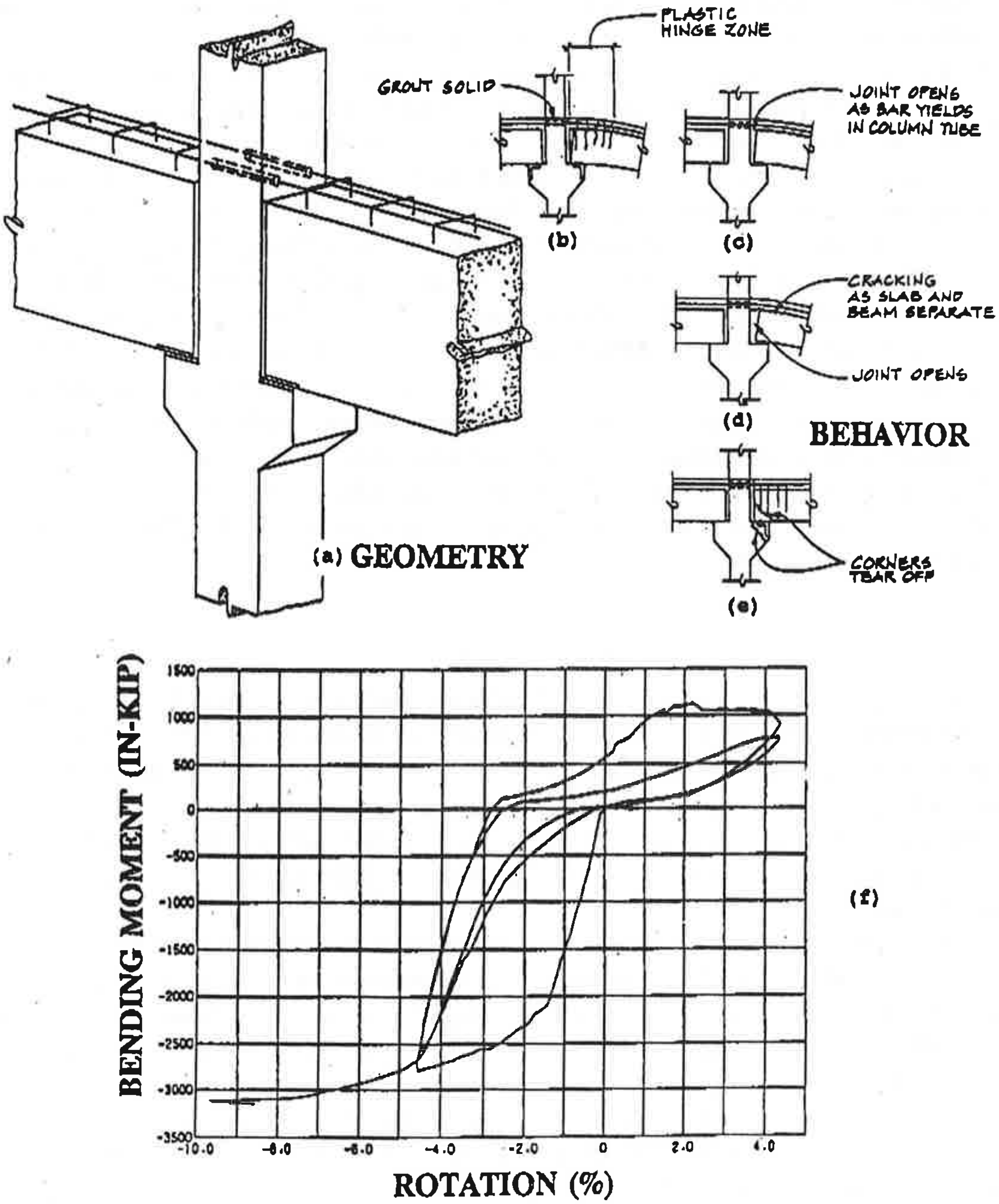


FIGURE C6A-1 Dry connections (PCI Research Project/4-86, Moment Connection BC16A).

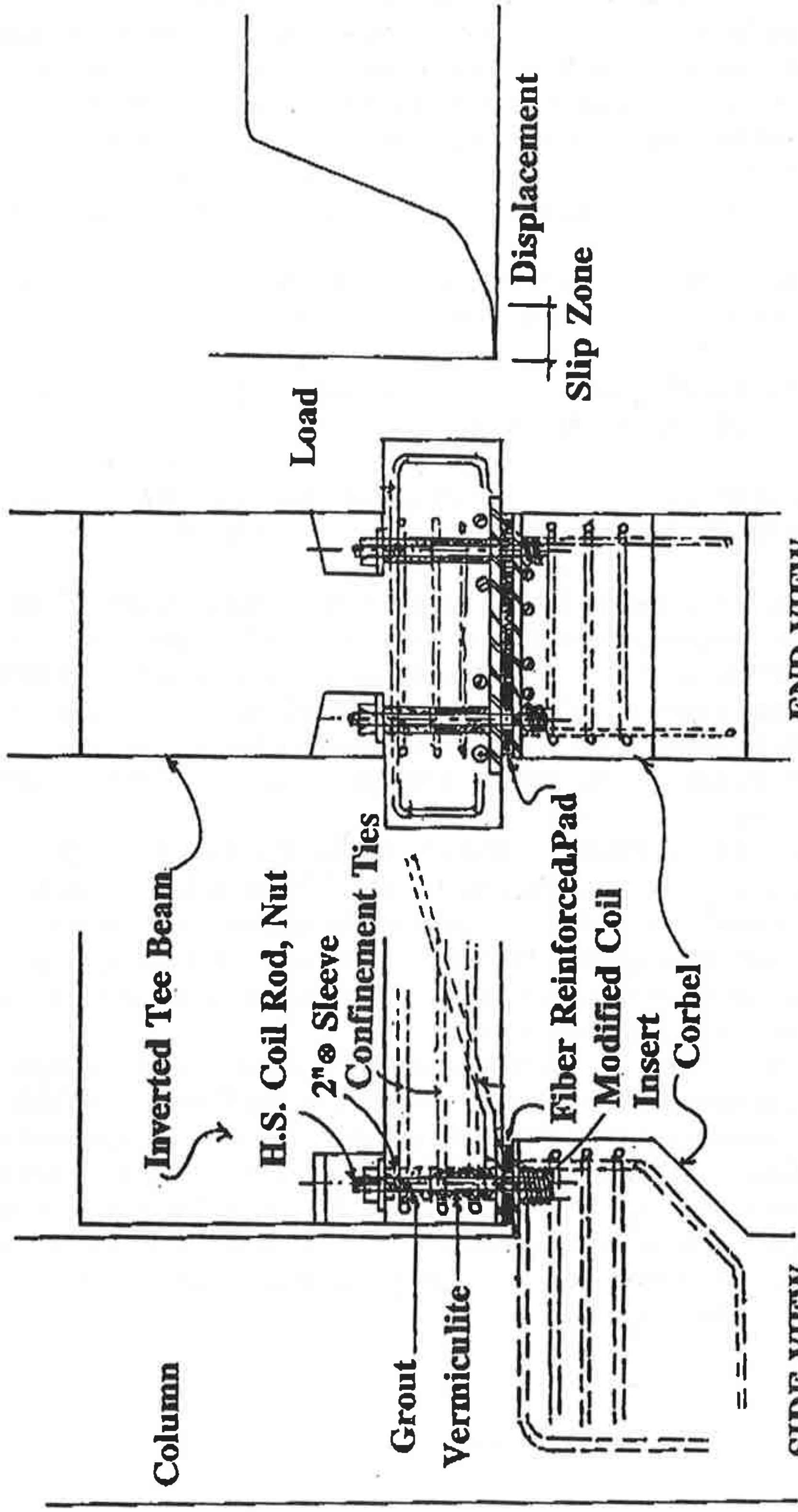


FIGURE C6A-2 Inverted tee beam to corbel connection (Shockey Bros., Winchester, Virginia).

It is essential that connection detailing recognize the load-deformation behavior imposed on connections by requirements of the seismic force resisting structural system selected and possible uncertainties in the connection's response. Ideally, appropriate detailing concepts should be developed through analytical modeling and verified by physical testing. Without physical testing, the uncertainties in the response make it imperative that the response modification coefficients and deflection amplification factors used for lateral-force-resisting systems constructed with dry connections be less than those for the comparable monolithic reinforced concrete structural systems. However, regardless of the R and C_d values used it is essential to identify:

1. The magnitude of the deformation demands to which each connection will be subjected in order for the structural system to achieve the overall deformation required of it;

2. The ability of each connection to provide that deformation and the associated probable strength without failure; and

3. The ability of each connection and the associated connection region to provide the necessary system stiffness and energy dissipation.

Sec. 6A.2 addresses identification of the relation between the deformation demand imposed on the connections, their deformation capacity, and the deformation demands, (R and C_d) selected for the structure. The designer is required to study those relationships and identify the potential for prying actions or undesirable rocking motions. Use of computer programs, such as Drain-2DX developed by Powell et al. (1993), usually will be necessary to study the relation between the deformation selected for the structure and the resultant deformation demands placed on the connections.

Sec. 6A.3 addresses limitations on the R and C_d factors that can be used for the lateral-force-resisting structural framing system. Values as large as those for monolithic construction can be used if the connection's response characteristics have been established by physical testing. If, however, those characteristics have been determined only through analytical modeling, values are required to be less than those for monolithic construction because of uncertainties about response of the actual connections.

For Table 6A.3.3, it is intended that the values selected for R_j and C_{dj} should have the same relation to one another as the values specified for R and C_d in Table 2.2.2. For example, for a special moment frame of reinforced concrete, R and C_d are specified in Table 2.2.2 as 8 and 5.5, respectively. For the same frame, connections of Category C are required by this appendix. Thus, by Table 6A.3.3, R_j can range for the precast frame from 4 to 7 and C_{dj} can range from 2.25 to 4.5. The footnote to Table 6A.3.3 requires that R_j and C_{dj} be varied in step so that the designer can choose, depending on the detailing practice used, R_j and C_{dj} values coupled as shown in Table C6A.3.3.

TABLE C6A.3.3 Restricted R and C_d for Connection Category C

Restricted Response Modification Coefficient, R_j	Restricted Deflection Amplification Factor, C_{dj}
4	2.25
5	3.00
6	3.75
7	4.5

Sec. 6A.4 provides a framework for evaluating connection performance. The connection is identified as having three factors contributing to its characterization: the connector, its anchorage, and the surrounding connection region. Three Connection Performance Categories are identified. For Connection Performance Category A, there are no special requirements but those connections can be used only for lateral-force-resisting systems of Seismic Performance Category A. Further, any dry connection for which there is not direct transfer of tensile or shear force from the connector's anchorage to the principal reinforcement of the precast element by welding, bolting, or adequate lap length must be assigned to Category A. Performance Category B connections must exhibit stable inelastic capacities with increasing reversed cyclic deformation demands. However, such connections do not have to have the energy dissipation hysterectic characteristics normally associated with monolithic concrete connections. For example, the connection illustrated in Figure C6A-3 has a satisfactory performance for a probable strength of 180 kips and a reversed deformation of 10 mm. A connection cannot be used for lateral-force-resisting systems of Seismic Performance Category C, D, or E unless that connection is of Connection Performance Category C for which the stressed area at the interface must be greater than 30 percent of that for the closest adjacent area of uniform stress in the precast element. This restriction is aimed at minimizing the concentration of inelastic deformations in the element by forcing connections to be designed so as to activate a significant volume of the adjacent precast element and, therefore, more closely replicate the behavior for monolithic reinforced concrete construction.

The majority of the requirements of this appendix are intended to apply to precast elements and nonlinear action location connections that are part of the lateral force resisting system. However, Sec. 6A.5.2 also addresses the special case where there is a connector that is on the lateral-load-resisting path and is not to be part of the nonlinear action response. Further, it is also very important to consider the integrity and flexibility of all other connections in the structure to determine that their behavior is compatible with the anticipated lateral movements of the building.

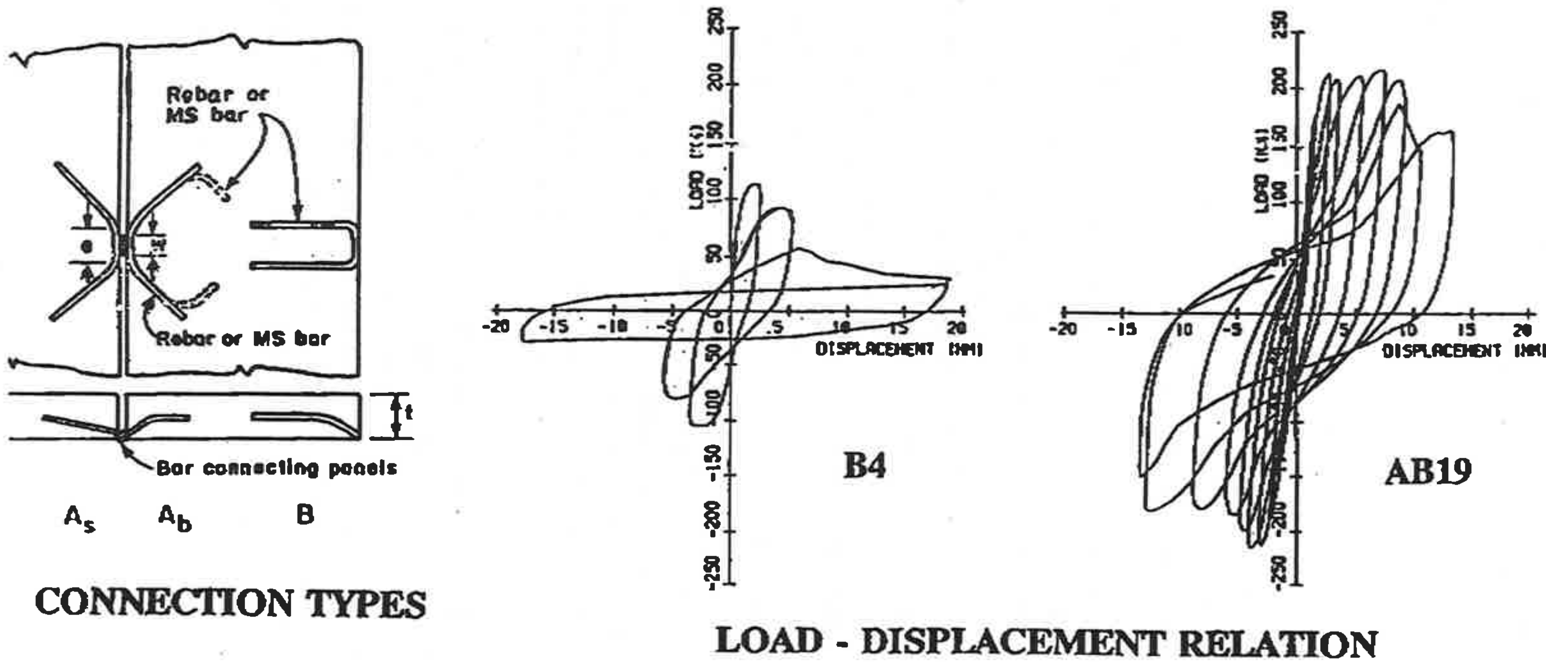


FIGURE 6A-3 Dry connections (R. Spencer, Earthquake Resistance Connections for Low Rise Precast Concrete Buildings, JSPS Seminar on Precast Concrete Construction in Seismic Zones).

REFERENCES

Applied Technology Council. 1981. *Proceedings of a Workshop on Design of Prefabricated Concrete Buildings for Earthquake Loads*, Report ATC-8.

Cheok, G. S., and H. S. Lew. 1991. "Performance of Precast Concrete Beam-to-Column Connections Subject to Cyclic Loading," *PCI Journal* 36 (3):56-67.

Clough, D. P. 1986. "A Seismic Design Methodology for Medium-Rise Precast Concrete Buildings." In *Proceedings of Seminar on Precast Construction in Seismic Zones*, JSPS/NSF, Tokyo, October.

Elliott, K. S., et al. 1987.

Hawkins, N. M., and R. E. Englekirk. 1987. "U.S.-Japan Seminar on P/C Concrete Construction in Seismic Zones," *PCI Journal* 32(2).

Jayashanker, V., and C. E. French. 1988. *An Interior Moment Resistant Connection Between Precast Elements Subjected to Cyclic Lateral Loads*, Structural Engineering Report 87-10. Mineapolis: University of Minnesota.

Mast, R. F. 1992. "A Precast Concrete Frame System for Seismic Zone Four," *PCI Journal* 37(1):50-64.

Nakaki, S. D., and R. E. Englekirk. 1991. "PRESS Industry Seismic Workshops: Concept Development," *PCI Journal* 36(5):54-61.

Neille, D. S. 1977. Behavior of Headed Stud Connections for Precast Concrete 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.

New Zealand Society of Earthquake Engineering. 1991. *Guidelines for the Use of Structural Precast Concrete in Buildings.*

Pekau, O. A., and D. Hum. 1991. "Seismic Response of Friction-Jointed Precast Panel Shear Walls," *PCI Journal* 36(2):56-71.

Powell, G., F. Filippou, V. Prakash, and S. Campbell. 1993. "Analytical Platform for Precast Structural Systems." In *Proceedings*, ASCE Structures Congress '93. New York: ASCE.

Priestley, M. J. N. 1991. "Overview of PRESS Research Program," *PCI Journal* 36(4):50-57.

Priestley, M. J. N., and J. R. Tao. 1992.

Stanton, J. F., R. G. Anderson, C. W. Dolan, and D. E. McClearly. 1986. *Moment-Resisting and Simple Connections*, Research Report 1/4, Prestressed Concrete Institute.

Stanton, J. F., T. R. Hicks, and N. M. Hawkins. 1991. PRESSS Project 1.3: Connection Classification and Evaluation. *PCI Journal* 36(5):62-71.

Chapter 7 Commentary

COMPOSITE STEEL AND CONCRETE STRUCTURE DESIGN REQUIREMENTS

7.1 SCOPE: This chapter represents the introduction in the United States of seismic design requirements for new composite (steel-concrete) structures. Due to a lack of design experience with certain types of composite structural systems under earthquake conditions, particularly in high seismic risk areas, many of these recommendations are necessarily of a conservative and qualitative nature. With further research and experience, it is expected that these provisions can be better quantified, refined, and expanded. The design and construction of composite elements and systems continues to evolve in practice. Nothing in these provisions is intended to limit the application of new systems, except where explicitly stated, provided that substantiating evidence based on tests and analysis is provided which demonstrates that the structure has adequate strength, ductility, and toughness for the intended purpose. Since many composite systems consist of assemblies of steel or reinforced concrete elements, Chapters 5 and 6 and the source documents referenced therein form an important basis to the Chapter 7 provisions.

7.2 REFERENCE DOCUMENTS: The provisions in this chapter are given in an ultimate strength limit state approach that is consistent with the loading criteria in Chapter 2 and the main AISC (Ref. 7-1 and 7-3) and ACI (Ref. 7-2) reference documents. Those portions of the reference documents that pertain to allowable stress design are excluded as a basis for these provisions.

7.3 DEFINITIONS AND SYMBOLS: Several of the definitions are based on Ref. 7-1 through 7.3. Others were developed specifically for this chapter.

7.4 COMPOSITE SYSTEMS: Because of limited experience with composite buildings subjected to extreme seismic forces, the designer is cautioned to give careful attention to all aspects of their design and particularly to the general building layout and detailing. However, available research does show that properly detailed elements and connections can perform as well under reverse cyclic loading as structural steel and reinforced concrete components. Throughout this commentary, many figures with composite connection details are shown. These details are only intended to show the basic character of the composite systems and should not be treated as standard details for design. Users are strongly encouraged to refer to the supporting references cited in the Commentary for more specific information on designing connections.

7.4.1 COMPOSITE PARTIALLY RESTRAINED FRAMES (C-PRF): Composite partially restrained frames consist of structural steel columns and composite steel beams connected together with semi-rigid composite connections (Zandonini and Leon, 1992). Semi-rigid composite connections are traditional steel frame connections in which the additional strength and stiffness provided by the floor slab has been incorporated by adding shear studs to the beams and slab reinforcement in the negative moment regions adjacent to the columns (see Figure C7.4.1).

This results in a more favorable distribution of strength and stiffness between negative and positive moment regions of the beams and provides for redistribution of forces under inelastic action.

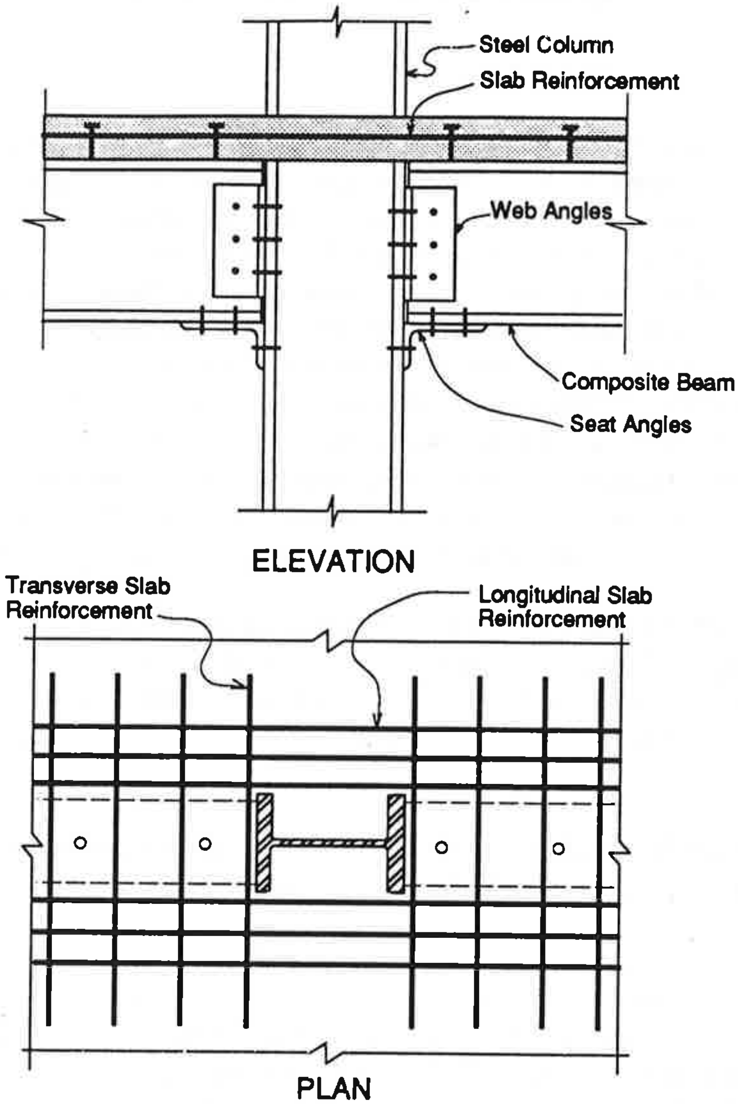


FIGURE C7.4.1 Composite partially restrained connection.

The design of the composite PR connections assumes that bending and shear forces can be considered separately with the bending assigned to a force couple formed by the steel in the slab and a bottom steel angle or plate and the shear assigned to a web angle or plate. Design methodologies for both C-PRF frames and connections have been published (Ammerman and Leon, 1990; Leon and Forcier, 1992; Steager and Leon, 1993), and standardized guidelines currently are being prepared by the ASCE Task Committee on Design Guide for Composite Semi-Rigid Connections.

Subassemblage tests show that when properly detailed, the partially restrained composite connections such as those shown in Figure C7.4.1 can undergo large deformations without fracturing. The connections generally are designed with a yield strength less than that of the connected members to prevent local failure modes (local buckling of the flange in compression, web crippling of the beam, panel zone yielding in the column, and bolt or weld failures). When these modes of failure are avoided, large connection ductilities should ensure excellent frame performance under large inelastic load reversals.

C-PRF were originally proposed for areas of low to moderate seismicity in the eastern United States (Seismic Performance Categories A, B, and C). However, with appropriate detailing and analysis, C-PRF can be used in areas of higher seismicity (Leon, 1990). Tests and analyses of these systems have demonstrated that the seismically induced forces on the partially restrained frames are lower than for ordinary frames due to: (a) lengthening in the natural period due to yielding in the connections and (b) stable hysteretic behavior of the connections. Similar findings also have been reported for steel frames with partially restrained connections (Nader and Astaneh, 1992; DiCorso et al., 1989). For these reasons, C-PRF may be designed for lower design forces than OMF, and in Table 2.2.2, C-PRF are assigned a R value of 6 that is between that of ordinary (R = 4-1/2) and special (R = 8) moment frames.

For frames up to four stories, the design should be made using an analysis that, as a minimum, accounts for the semi-rigid behavior of the connections by utilizing linear springs with reduced stiffness (e.g., see Bjorhovde, 1984). The effective connection stiffness should be considered for determining member force distributions and deflections, calculating the building's period of vibration, and checking frame stability. Frame stability can be addressed using conventional effective buckling length procedures (Ref. 7-1); however, the connection flexibility must be considered in determining the rotational restraint at the ends of the columns. For structures taller than four stories, drift and stability need to be carefully checked using analysis techniques that incorporate both geometric and connection non-linearities (Liu and Chen, 1984; Ammerman and Leon, 1990). Semi-rigid composite connections also can be used as part of the gravity load system for braced frames provided that minimum design criteria such as those proposed by Leon and Ammerman (1990) are followed; in this case, no height limitation applies, and the frame should be designed as a braced system following the requirements in Chapter 5 and 7, as applicable.

Because the moments of inertia for composite beams in the negative and positive regions are different, the use of either value alone for the beam members in the analysis can lead to significant errors. Therefore, the use of a weighted average is recommended (Leon and Ammerman, 1990; Ammerman and Leon, 1990; Zaremba, 1988).

7.4.2 COMPOSITE ORDINARY MOMENT FRAMES (C-OMF): C-OMF include a variety of configurations where steel or composite beams are combined with either steel, reinforced concrete, or composite columns. In particular, C-OMF with reinforced concrete or composite columns have been used in recent years as a cost-effective alternative to frames with steel columns (Griffis, 1992). In most C-OMF, it is anticipated that much of the inelastic deformations will occur in the steel beams; therefore, in Table 2.2.2, C-OMF are given the same R and C_d values and are subject to the same use restrictions as steel OMF.

Examples of connections used in C-OMF are shown in Figures C7.4.2. Since the late 1980s, over 60 large-scale tests of the connections shown in Figure C7.4.2-1 have been conducted in the United States and Japan under both monotonic and cyclic loading (Sheikh et al., 1989; Kanno, 1993; Nishiyama et al., 1990). The results of these tests show that carefully detailed connections can perform as well as seismically designed steel or reinforced concrete connections. One important finding in the research is the effectiveness of face bearing plates (see Figure C7.4.2-1) for mobilizing the shear strength of reinforced concrete in the joint region. Further information on design methods and equations for these composite connections has been reported by Deierlein et al. (1989) and the ASCE Committee on Design Criteria for Composite Steel-Concrete Structures (ASCE, 1994). Note that while the scope of the current ASCE Guidelines is limited to regions of low- to moderate-seismicity, recently reported work by Kanno (1993) indicates that the ASCE guidelines are adequate for regions of high seismicity as well.

Connections between steel beams and composite columns (Figure C7.4.2-2) have been used and tested extensively in Japan and established design provisions are included in the Architectural Institute of Japan's *Standards for Structural Calculation of Steel Reinforced Concrete Structures* (1987). In this case, the connection strength may be conservatively calculated as the strength of the steel beam to steel column connection. In certain cases, additional strength can be calculated considering the concrete encasement using design models for steel beam to reinforce concrete column connections

Connections to filled composite columns (Figure C7.4.2-3) have been used less frequently and only a few tests of these types have been reported (Azizinamini and Prakush, 1993). Where the steel beam members are continuous through the composite column, the internal force transfer mechanisms and behavior of these connections are similar to those for connections to reinforced concrete columns (Figure C7.4.2-1).

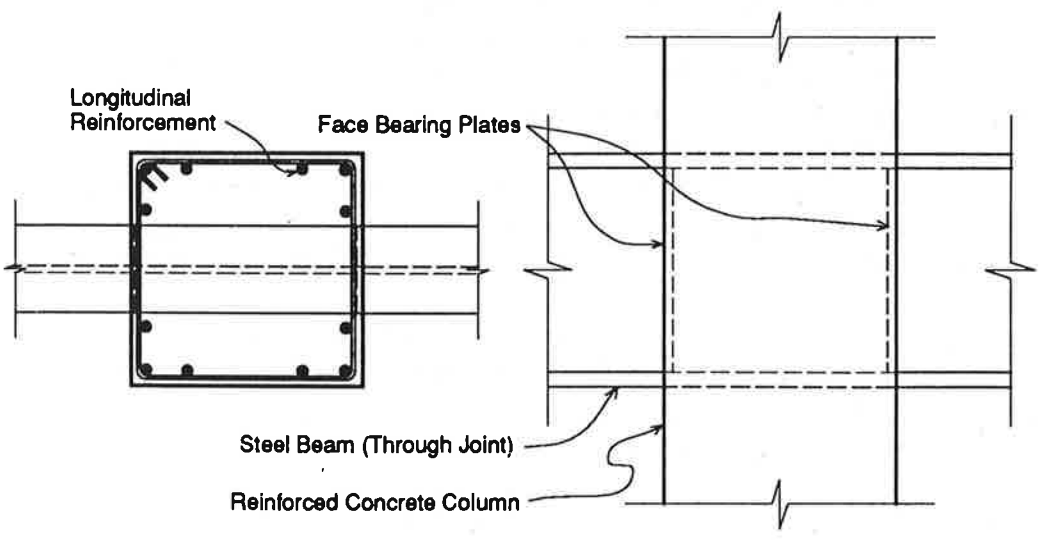


FIGURE C7.4.2-1 Reinforced concrete column-to-steel beam connection.

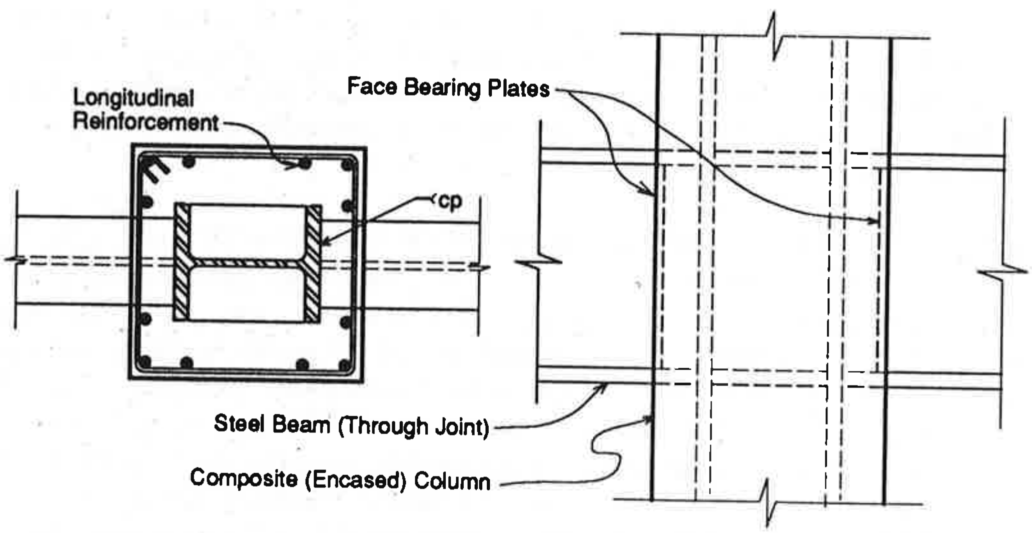


FIGURE C7.4.2-2 Composite (encased) column-to-steel beam connection.

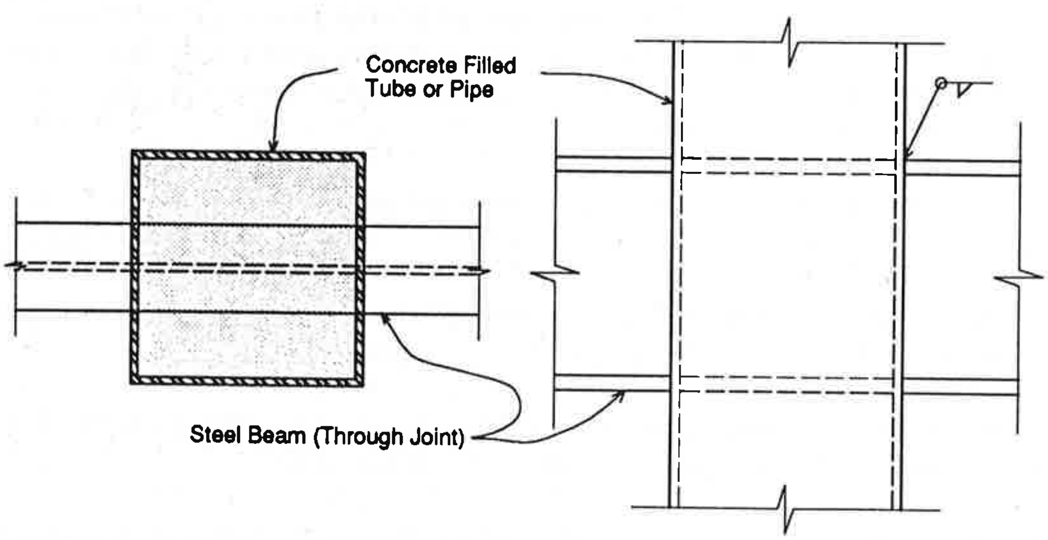


FIGURE C7.4.2-3 Concrete filled tube column-to-steel beam connection.

Design requirements for steel and composite beams, steel columns, and composite moment connections are based on those for OMF of structural steel as specified in Chapter 5 and Ref. 7-1 and 7-3. Provisions and commentary regarding the design requirements for composite columns are presented in Sec. 7.5.3 and 7.5.4.

Design and detailing requirements for reinforced concrete columns are equivalent to those in Chapter 6 and Ref. 7-2 for either intermediate or special moment frames of reinforced concrete. The provisions distinguish between intermediate versus special moment frame detailing requirements to match the minimum requirements imposed on reinforced concrete columns in the various Seismic Performance Categories (i.e., intermediate frame requirements are used in SPC A, B, and C and special frame requirements are used in SPC D and E). The requirement for special moment frame detailing in Seismic Performance Categories D and E is a conservative measure that was decided upon in the absence of research that would permit the use of intermediate detailing of reinforced concrete columns in high seismic regions. As more research and experience become available, it is anticipated that this requirement may be relaxed since the beams and beam-column joint details in composite frames generally perform better under cyclic loads than those of reinforced concrete.

7.4.3 COMPOSITE SPECIAL MOMENT FRAMES (C-SMF): The provisions for C-SMF are similar to those for C-OMF except that more stringent design and detailing provisions are imposed to provide greater toughness in the members and connections, thereby allowing the use of lower design forces. Generally, the provisions for C-SMF are designed to confine inelastic hinging to the beams and to minimize damage to the columns and connections. Tests reported by Kanno (1993) demonstrate that it is quite feasible and effective to design composite beam-column connections where little damage to the connection occurs under large cyclic hinging of the beam adjacent to the connection. Since the inelastic behavior of C-SMF is at least comparable to that for steel or reinforced concrete SMF, the R and C_d values for C-SMF in Table 2.2.2 are the same as for those systems.

The major differences in the provisions between C-OMF and C-SMF are that in C-SMF:

a. Steel and reinforced concrete columns are required to meet the design provisions for SMF (from Chapters 5 and 6, respectively) in all Seismic Performance Categories. As described in the commentary to Sec. 7.5.3 and 7.5.4, the detailing requirements for composite columns also are more stringent for C-SMF.

b. Steel and composite beams are required to meet the more restrictive section slenderness criteria (b/t and h/tw ratios) and lateral bracing criteria used for steel SMF (Chapter 5).

c. Beam-column moment connections are required to develop the strength of the connected beams following the criteria used for steel SMF (Chapter 5).

d. The strong-column/weak-beam philosophy is followed using provisions similar to those for reinforced concrete SMF (Chapter 6) or steel SMF (Chapter 5).

The use of steel or composite trusses as flexural members in C-SMF is not permitted unless substantiating evidence is provided to demonstrate adequate seismic resistance of the system. This limitation applies only to members that are part of lateral-force-resisting frames and does not apply for joists and trusses that carry gravity loads only (i.e., this restriction does not apply to the majority of applications where simply supported floor joists and trusses are used as simple framing members to support the floor system). Trusses and open web joists generally are regarded as ineffective as flexural members in lateral load systems unless either (a) the web members have been carefully detailed through a capacity design approach to delay, control, or avoid overall buckling of compression members, local buckling, or failures at the connections (Itani et al., 1991) or (b) a strong-beam/weak-column mechanism is adopted and the truss and its connections proportioned accordingly (Camacho et al., 1993). Both approaches can be used for one-story industrial-type structures where the gravity loads are small and ductility demands on the critical members can be sustained. Under these conditions and when properly proportioned, these systems have been shown to provide adequate ductility and energy dissipation capacity.

7.4.4 COMPOSITE CONCENTRICALLY BRACED FRAMES (C-CBF): Concentrically braced frames with composite elements have been used in low- and high-rise buildings in regions of low and moderate seismicity. Experience is limited in high seismic regions; however, the design provisions for C-CBF are intended to result in behavior comparable to steel CBF where the braces often are the elements most susceptible to inelastic deformations (e.g., see the commentary to Sec. 9 of Ref. 7-3). Therefore, the R and C_d values in Table 2.2.2 for C-CBF are the same as those for steel CBF.

In cases where composite braces are used (either concrete filled or concrete encased), the concrete has the potential to stiffen the steel section and prevent or deter brace buckling while at the same time increasing the capacity to dissipate energy. The filling of steel tubes with concrete has been shown to effectively stiffen the tube walls and inhibit local buckling (Goel and Lee, 1992). For concrete encased steel braces, the concrete must be sufficiently reinforced and confined to prevent the steel shape from buckling. It is recommended that composite braces be designed to meet all requirements of composite columns as specified in this chapter. Composite braces in tension should be designed based on the steel section alone unless test data justify higher strengths. Braces that are all steel should be designed to meet all requirements for steel braces in Chapter 5. Reinforced concrete and composite columns in CBF are detailed to provide comparable ductility to that provided in C-OMF. This should be conservative since the design loads in CBF are higher than those in OMF and the ductility demands generally are less. With further research, these detailing requirements may be relaxed.

Examples of connections used in C-CBF are shown in Figures C7.4.4-1 through C7.4.4-3. Careful design and detailing of the joints in a C-CBF are required to prevent failure before developing the strength of the braces in either tension or compression. All connection strengths should be capable of developing the full strength of the braces in tension and compression. Where the brace is composite, the added brace strength afforded by the concrete should be considered. In such cases, it would be unconservative to base the connection strength on the steel section alone. Connection design and detailing should recognize the fact that buckling of the brace could cause excessive rotation at the brace ends and lead to local connection failure.

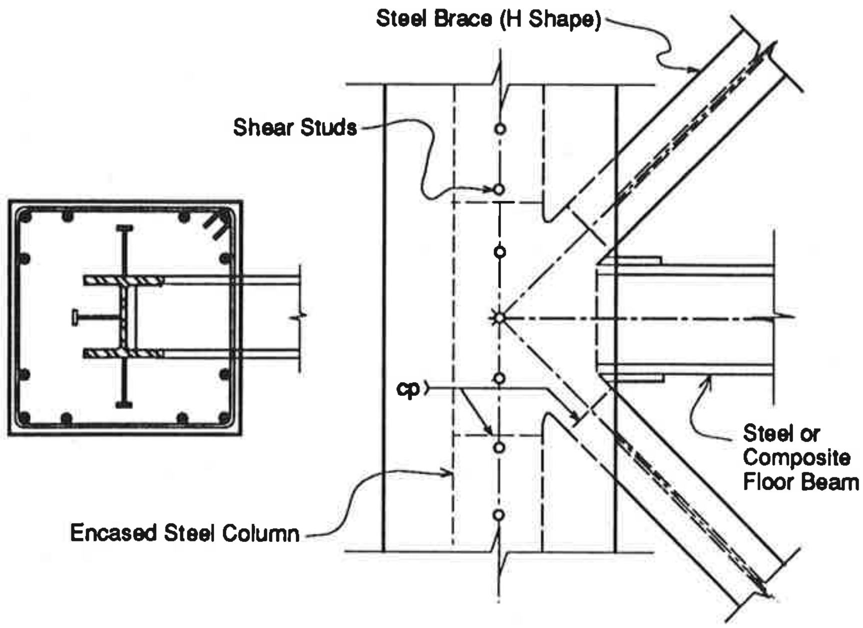


FIGURE C7.4.4-1 Reinforced concrete (or composite) column-to-steel concentric brace.

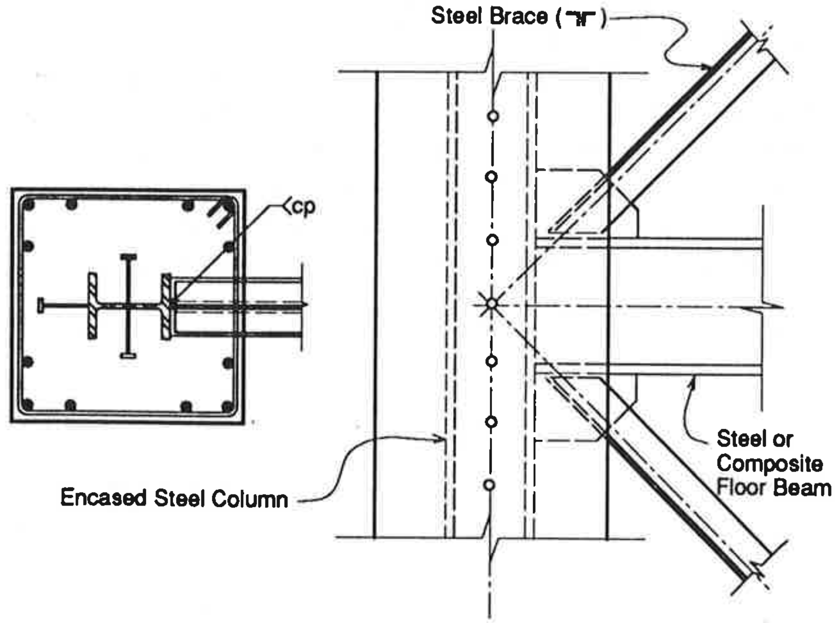


FIGURE C7.4.4-2 Reinforced concrete (or composite) column-to-steel concentric brace.

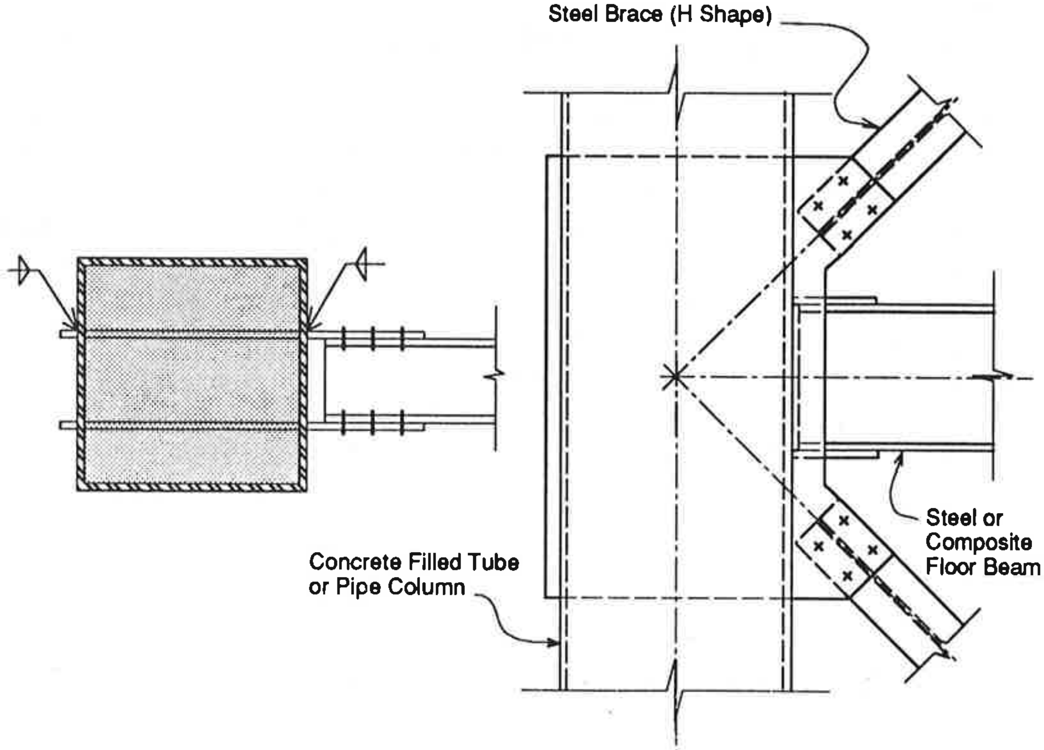


FIGURE C7.4.4-3 Concrete filled tube or pipe column-to-steel concentric brace.

7.4.5 COMPOSITE ECCENTRICALLY BRACED FRAMES (C-EBF): All-steel EBF have been extensively tested and utilized in California and generally are recognized as providing excellent resistance and energy absorption for seismic loads (see commentary to Sec. 10 in Ref. 7-3). While little experience exists concerning the use of C-EBF, the behavior of the critical steel link beam should be essentially the same as for steel EBF and inelastic deformations in the composite or reinforced concrete columns should be minimal. Therefore, the R and C_d values for C-EBF are the same as those for steel EBF. As described below, careful design and detailing of the brace-to-column and link beam-to-column are essential to the design.

The basic requirements for the C-EBF are the same as those for steel EBF with additional provisions for the design of reinforced concrete or composite columns and the composite connections. Since inelastic deformations of the columns should be small, the basic detailing provisions for reinforced concrete and encased composite columns are based on requirements for intermediate reinforced concrete frames. Additionally, as a conservative measure until there is further research and more experience with C-EBF, reinforced concrete and composite columns in Seismic Performance Categories D and E must meet more stringent transverse reinforcement provisions equivalent to those for reinforced concrete SMF. These more stringent transverse reinforcement provisions also are required in all Seismic Performance Categories where the shear link is adjacent to the column. This requirement is in recognition of the large moments and force reversals imposed in the columns near the links.

Satisfactory behavior of EBF are dependent on making the braces and columns strong enough to remain essentially elastic under forces generated by inelastic deformations of the links. Since this requires an accurate calculation of the shear link strength, it is important that the shear link region of the link beam not be encased in concrete. Portions of the link beam outside of the link region may be encased since an over-strength outside the link would not reduce the effectiveness of the system. Link regions may be composite with the slab since the slab has a minimal effect on the strength of links that undergo shear yielding. The additional strength provided by composite action with the slab is, however, important to consider for long links where their design shear strength is governed by flexural yielding at the ends of the link (Ricles et al., 1989).

In C-EBF where the shear link is not adjacent to the column, the concentric connections between the braces and columns are similar to those in C-CBF (see Figure C7.4.4-1 through C7.4.4-3). An example where the shear link is adjacent to the column is shown in Figure C7.4.5. In this case, the steel link to column connection is similar to composite beam-column moment connections in C-SMF (see Sec. 7.4.3) and to steel coupling beam-wall connections (see Sec. 7.4.6). The following references provide guidance for connections similar to that in Figure C7.4.5: Harries et al., 1993; Shahrooz et al., 1993; Sheikh et al., 1989; Deierlein et al., 1989; and Kanno, 1993.

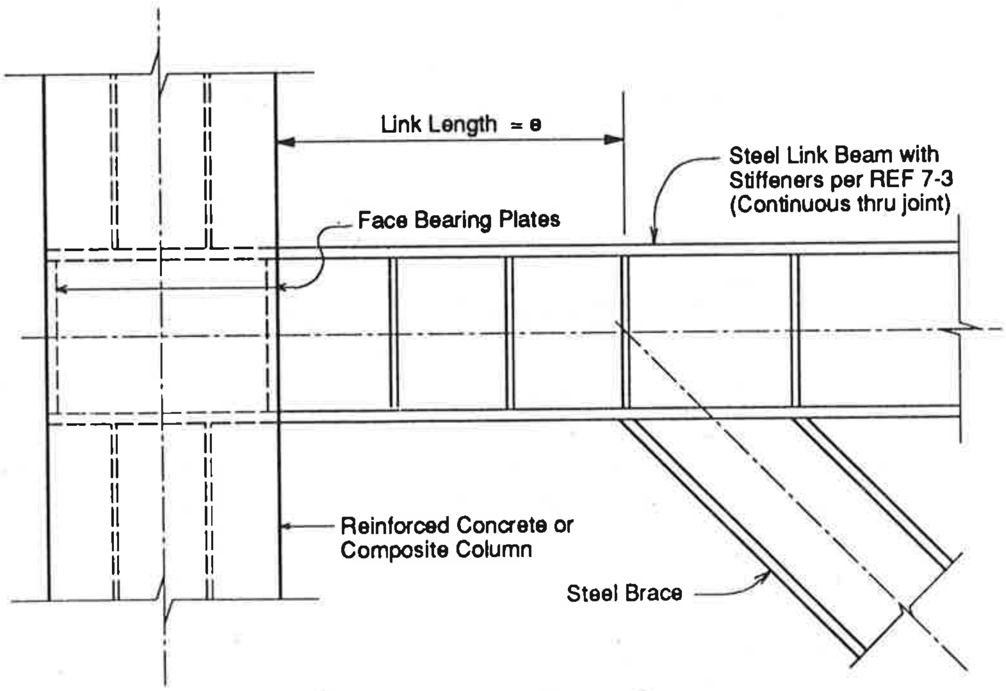


FIGURE C7.4.5 Reinforced concrete (or composite) column-to-steel eccentric brace.

7.4.6 RC WALLS COMPOSITE WITH STEEL ELEMENTS: The provisions in this section apply to three variations of structural systems using reinforced concrete walls. One type is where reinforced concrete walls are used as infill panels in what are otherwise steel or composite frames. Examples of typical sections at the wall to column interface for such cases are shown in Figures C7.4.6-1 and C7.4.6-2. The details in Figure C7.4.6-2 also may occur in the second type of system where encased steel sections are used as vertical reinforcement in what are otherwise reinforced concrete shear walls. Finally, the third variation is where steel or composite beams are used to couple two or more reinforced concrete walls. Examples of the wall to coupling beam connection are shown in Figures C7.4.6-3 and C7.4.6-4. When properly designed, each of these systems should have the strength and toughness at least equal to that of pure reinforced concrete shear wall systems. Therefore, the R and C_d values given in Table 2.2.2 (5-1/2 and 5, respectively) are the same as those for reinforced concrete shear wall systems.

For cases where the reinforced concrete walls frame into non-encased steel shapes (Figure C7.4.6-1), mechanical connectors are required to transfer vertical shear between the wall and column, and to anchor the wall reinforcement. Additionally, if the wall elements are interrupted by steel beams at floor levels, shear connectors are needed at the wall-to- beam interface. Tests on concrete and masonry infill walls have shown that if shear connectors are not present, story shear forces are carried primarily through diagonal compression struts in the panel (Chrysostomou, 1991). This behavior often induces high forces in localized areas of the walls, beams, columns, and connections. The shear stud requirements (Sec. 7.4.6.2.3) will improve performance by providing a more uniform transfer of forces between the infill panels and the boundary members. As a conservative measure until further research data are available, the provisions require that a reduced value of the shear stud strengths be used in the higher Seismic Performance Categories. This is done because existing provisions for calculating the nominal strength of shear studs (e.g., Ref. 7-1) are based on static tests. This reduction in stud strengths does not apply to cases where the steel member is fully encased since the provisions conservatively neglect the contribution of bond and friction between the steel and concrete.

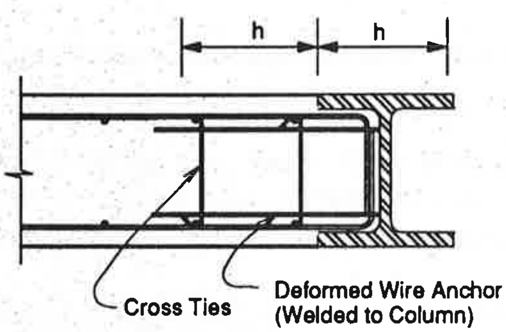


FIGURE C7.4.6-1 Partially encased steel boundary element.

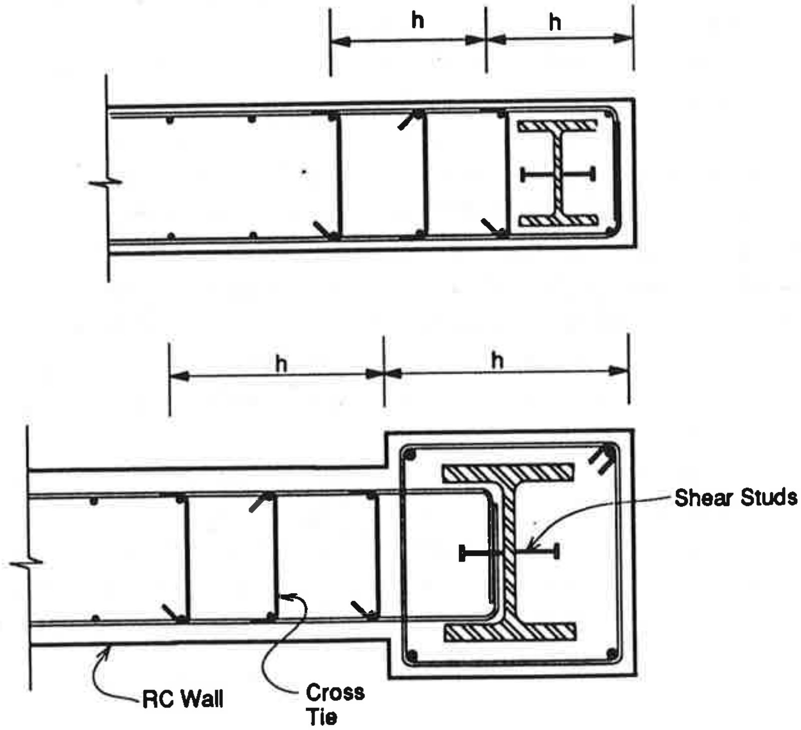


FIGURE C7.4.6-2 Fully encased composite boundary element.

Some evidence exists that walls with encased steel boundary members may have a tendency to split along vertical planes inside the wall near the column. Therefore, the provisions require that transverse steel be continued into the wall for the distance $2h$ as shown in Figures C7.4.6-1 and C7.4.6-2.

Two examples of connections between steel coupling beams to concrete walls are shown in Figures C7.4.6-3 and C7.4.6-4. Under high seismic loads, the coupling beams are likely to undergo large inelastic deformations through either flexural and/or shear yielding. Therefore, the coupling beams are required to satisfy many of the requirements for link beams in EBF. It should be noted, however, that the shear link requirements for steel EBF are intended for unencased steel members. For encased coupling beams, it may be possible to reduce the web stiffener requirements of Sec. 10.3.b of Ref. 7-3, but currently, there are no data available that provides design guidance on this.

The requirements for coupling beams and their connections are based largely on recent tests of unencased steel coupling beams by Shahrooz et al. (1993) and Harries et al. (1993). Their test data and analyses show that properly detailed coupling beams can be designed to yield at the face of the concrete wall and provide stable hysteretic behavior under reverse cyclic loads. Aside from the work of Shahrooz and Harries, supporting information related to the design of the steel beam to wall connection is included in the work cited in Sec. C7.4.2 for moment connections between steel beams and reinforced concrete columns.

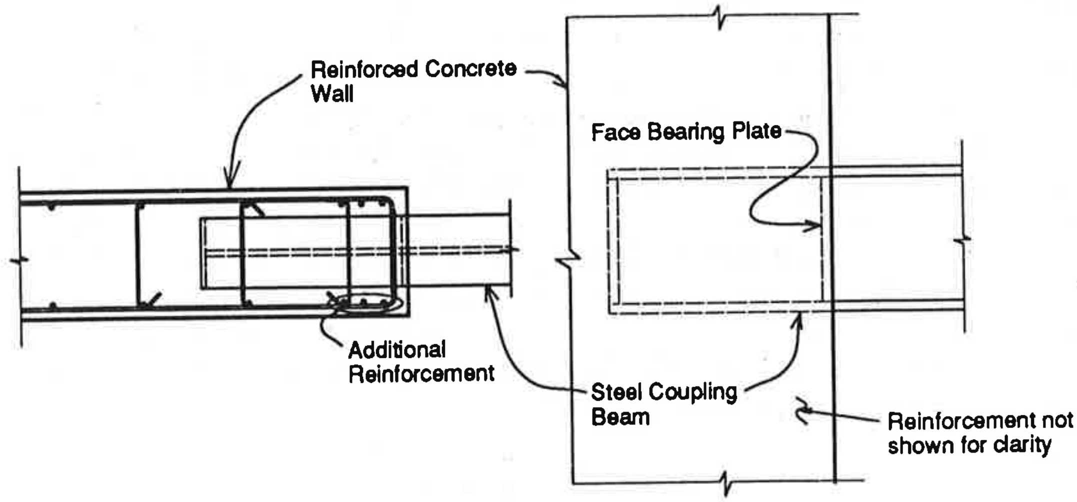


Figure C7.4.6-3 Steel coupling beam to reinforced concrete wall.

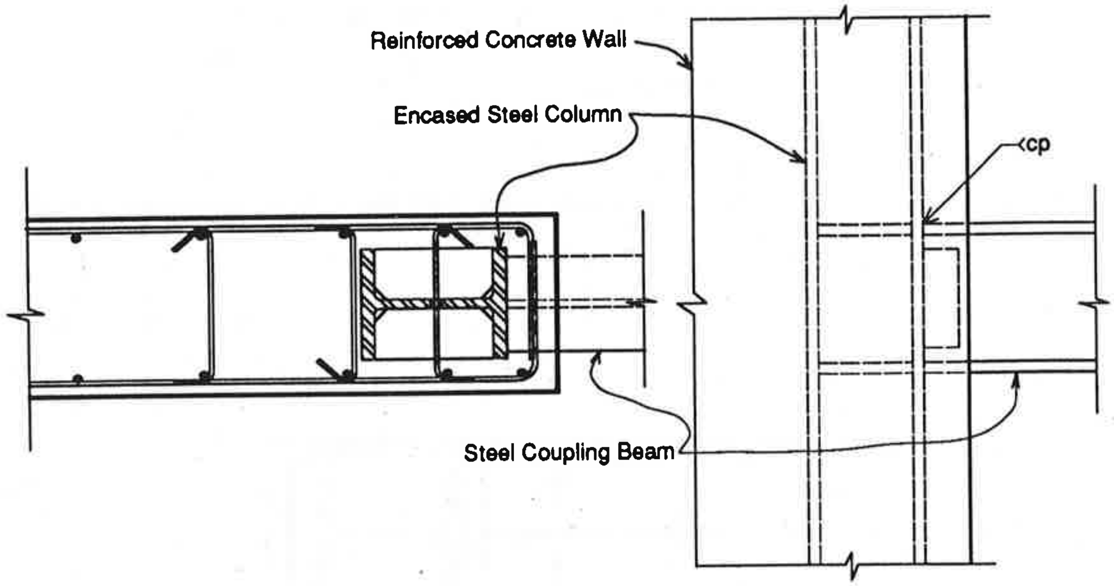


Figure C7.4.6-4 Steel coupling beam to reinforced concrete wall with composite boundary member.

7.4.7 STEEL PLATE REINFORCED COMPOSITE SHEAR WALLS: Steel plate reinforced composite shear walls usually are advantageous where story shear forces are large and the required thickness of conventionally reinforced shear walls is excessive. The provisions limit the shear capacity of the wall to the yield strength of the plate because there is insufficient evidence to develop design rules for combining the yield strength of the steel plate and the reinforced concrete panel. Also, since the shear strength of the steel plate usually is much greater than that of the reinforced concrete encasement, neglecting the contribution of the concrete does not have a significant practical impact. Structures with composite walls are assigned a higher R value than those with reinforced concrete walls (R = 6-1/2 vs. 5-1/2) because the shear yielding mechanism of the steel plate will result in more stable hysteretic loops than for reinforced concrete walls. The value of R = 6-1/2 is also the same as that for light frame walls with shear panels.

Two examples of connections between composite walls to either steel or composite boundary elements are shown in Figures C7.4.7-1 and C7.4.7-2. The provisions require that the connections between the plate and the boundary members (columns and beams) be designed to develop the full yield strength of the plate. Minimum reinforcement in the concrete cover is required to maintain the integrity of the wall under reverse cyclic loading and out-of-plane forces. Until further research data are available, the minimum required wall reinforcement is based on the minimum specified value for reinforced concrete walls in Ref. 7-2.

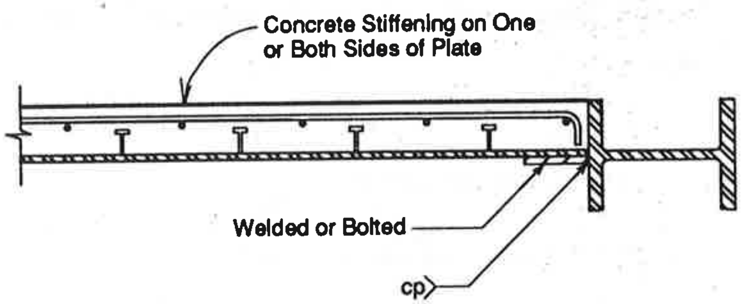


FIGURE C7.4.7-1 Concrete stiffened steel shear wall with steel boundary member.

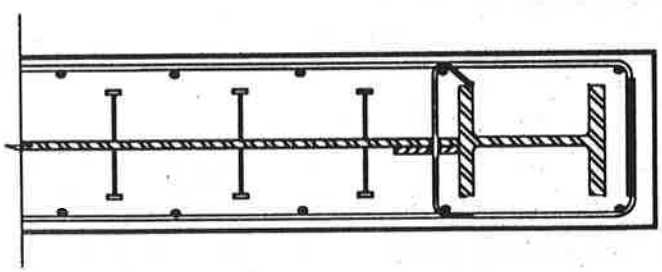


FIGURE C7.4.7-2 Concrete stiffened steel shear wall with composite (encased) boundary boundary member.

The thickness of the concrete encasement and the spacing of stud shear connectors should be calculated to ensure that the plate can reach yield prior to overall or local buckling. It is recommended that overall buckling of the composite panel be checked using elastic buckling theory using a transformed section stiffness of the wall. For plates with concrete on only one side, stud spacing requirements that will meet local plate buckling criteria can be calculated based on h/t provisions for the shear design of webs in steel girders. For example, in Sec. F2.2 of Ref. 7-1 the limiting h/t value specified for compact webs subjected to shear is $h/t \le 187\sqrt{k/Fy}$ where F_y is in ksi units ($h/t \le 491\sqrt{k/F_y}$ where F_y is in MPa units). Assuming a conservative value of the plate buckling coefficient $k = 5$ and $Fy = 50$ ksi (344.5 MPa), this equation gives the limiting value of $h/t \le 59$. For a 3/8 in. (9.5 mm) thick plate this gives a maximum value of $h = 22$ in. (559 mm) that is representative of the maximum center-center stud spacing that should suffice for the plate to reach its full shear yielding strength.

Careful consideration should be given to the shear and flexural strength of wall piers and of spandrels adjacent to openings. In particular, composite walls with large door openings may require structural steel boundary members attached to the steel plate around the openings.

7.5 COMPOSITE MEMBERS: The material limitations for composite members of steel and concrete are based on the values cited in Ref. 7-1 to 7.3 for structural steel, reinforced concrete and composite structures.

7.5.1 COMPOSITE SLABS: Out-of-plane shear and flexural strength design procedures for composite metal deck slabs are well established and documented in Ref. 7-5. In addition, publications by the Steel Deck Institute and its members (Steel Deck Institute, 1993; Vulcraft, 1990) provide a code-recommended standard practice, specification and load tables.

In many cases of practical design, composite metal deck slabs are relied upon to provide in-plane diaphragm strength for distributing seismic forces to the lateral- force-resisting system. Recent tests by Luttrell at West Virginia University have provided the basis for a rational design procedure published by Steel Deck Institute (1987).

Steel deck diaphragms may be reinforced with overlayments of insulating concrete or structural concrete or by directly attached panels used to provide a flat surface. Such practice presents additional paths through which shear forces may be transferred across the diaphragm. The concrete that fills the corrugations prohibits or limits end warping and local corner buckling thereby increasing both strength and stiffness provided that sufficient perimeter attachments are added to transfer forces across the diaphragm perimeters. As concrete thickness over the steel deck increases, shear strength can approach that for a reinforced flat slab of the same thickness. For composite floor deck diaphragms having cover depths between 2 in. (51 mm) and 6 in. (152 mm), measured shear stresses on the order of $3.5\sqrt{f_c}$ (where $\sqrt{f_c}$ and f_c' are in psi units) over the shear area (using the concrete cover thickness over the deck) have been reported.

The design provisions of Ref. 7-5 are based on nominal temperature and shrinkage reinforcement in the concrete slab. Note that manufacturers of metal deck publish information on composite slab diaphragms that may be used in design (Vulcraft, 1990). Naeim (1989) and the Steel Deck Institute (1987) provide examples of diaphragm designs. Recent research on diaphragms has also been reported by Easterling and Porter (1993).

The diaphragm strength of concrete metal deck slabs also can be based on the principles of reinforced concrete design using the concrete and reinforcement above the metal deck ribs and ignoring the beneficial effect of the concrete filled flutes. Care must be taken to insure that the shear forces can be transferred either through welds and/or shear devices in the collector and boundary elements.

It is advisable to use mechanical shear devices such as headed studs to transfer diaphragm forces between the slab and collector/boundary elements, particularly in complex shaped diaphragms with discontinuities. However, in low-rise buildings without abrupt discontinuities in shape of the diaphragm or in the lateral-force-resisting system, the standard metal deck attachment procedures can usually suffice.

7.5.2 COMPOSITE BEAMS: These provisions apply only to composite beams in frames that are part of the primary lateral-force-resisting system. The provisions do not apply to composite beams that are designed for gravity loads only. In general, the provisions follow those for composite beams in Ref. 7-1.

Equation 7.5.2.2 is to insure that in C-SMF the steel strain at the extreme fiber will be at least five times the tensile yield strain prior to concrete crushing at strain equal to 0.003. This ductility limit is expected to control the beam geometry in only extreme beam/slab proportions.

While the Provisions permit composite beams to be designed according to Ref. 7-1, the effects of reverse cyclic loading on the strength and stiffness of shear studs should be considered. This is particularly important for C-SMF where the design forces are calculated assuming large member ductility and toughness. In the absence of test data to support specific requirements in the Provisions, the following are suggested as special measures to consider for the design and installation of shear studs in C-SMF: (a) implementation of an inspection and quality assurance plan to insure proper welding of shear studs to the beams, and (b) use of additional studs beyond those required by Ref. 7-1 in regions of the beams where plastic hinging is expected. Although no test data are available to support the latter suggestion, members of the NEHRP update committee have recommended that the values of nominal stud strengths in Ref. 7-1 be reduced by 10 to 25 percent where studs undergo large reverse cyclic loading.

7.5.3 ENCASED COMPOSITE COLUMNS: The minimum ratio of the structural steel to column area of A_s/A_g equal to 4 percent follows that of Ref. 7-1. Detailed requirements for reinforcing bar details of composite columns that are not covered in Ref. 7-1 are included in these provisions based on Ref. 7-2.

As with any column of concrete and reinforcing steel, the designer must be keenly aware of the potential problems in reinforcing steel placement and congestion as it affects the constructability of the column. This is particularly true at beam column joints where potential interference between a steel spandrel beam, a perpendicular floor beam, vertical bars, joint ties, and shear connectors can cause difficulty in reinforcing bar placement and a potential for honeycombing of the concrete.

Composite columns can be an ideal solution for use in seismic regions because of their inherent structural redundancy. If a composite column is designed so that the structural steel portion carries most or all of the dead load acting alone, then an extra degree of protection and safety is afforded for public safety, even in a severe earthquake where excursions into the inelastic zone can be expected to deteriorate concrete cover and buckle reinforcing steel.

7.5.3.1 Shear Strength: Specific instructions are given for the determination of shear capacity in concrete encased steel composite members including assignment of some shear to the reinforced concrete encasement. Examples for determining the effective shear width, b_w, of the reinforced concrete encasement are illustrated in Figure C7.5.3.1. These provisions exclude any strength, V_c, assigned to concrete alone following recommendations by Furlong (1988).

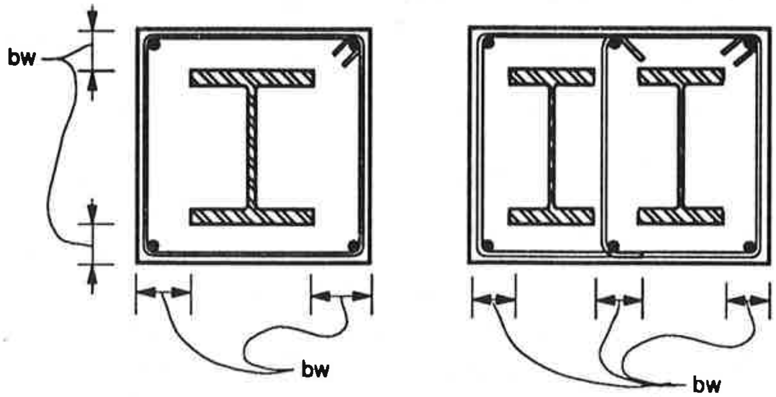


FIGURE C7.5.3.1 Effective widths for shear strength calculation of encased composite columns.

7.5.3.2 Shear Connectors: Currently no existing specifications in the United States include requirements for shear connectors for encased steel sections. The provisions in Sec. 7.5.3.2 should be conservative since they require that shear connectors be provided to transfer all calculated forces between the structural steel and the concrete and the contribution of bond and friction is neglected. Further suggestions and information regarding the design of shear connectors for encased members are provided by Griffis (1992) and AISC (1992).

7.5.3.3 Transverse Reinforcement: The tie requirements in this section are essentially the same as those for composite columns as specified in Sec. 10.14.8 of Ref. 7-2.

7.5.3.4 Longitudinal Reinforcement: The requirements for longitudinal bars are essentially the same as those that would apply to composite columns in low seismic zones as specified in Ref. 7-2. The distinction between load carrying and restraining bars is made to allow for longitudinal bars (restraining bars) that are provided solely for erection purposes and to improve confinement of the concrete. Due to interference with steel beams framing into the encased members, the restraining bars are often discontinuous at floor levels and, therefore, are not used to resist longitudinal forces in the column.

7.5.3.5 Steel Core: The requirements for the steel core are essentially the same as those for composite columns as specified in Ref. 7-1 and 7-2.

7.5.3.6 Additional Requirements in Seismic Performance Category C: The more stringent tie spacing requirements follow those for reinforced concrete columns in regions of "moderate seismicity" that translate into the requirements for intermediate reinforced concrete frames in Chapter 6. These are applied to all composite columns in Seismic Performance Category C and above to make the composite column details at least equivalent to the minimum level of detailing required by Chapter 6 for reinforced concrete columns in IMF which are permitted in Seismic Performance Category C. The tie requirements are based on Sec. 21.8.5 of Ref. 7-2.

7.5.3.7 Additional Requirements in Seismic Performance Categories D and E: Additional requirements for composite columns are based on those required in Chapters 5 and 6 for steel and reinforced concrete columns in Seismic Performance Categories D and E. For additional commentaries, see Chapters 5 and 6 and the cited sections of Ref. 7-2 and 7-3.

The minimum tie area requirement of Eq. 7.5.3.7.2 is based on that required in Ref. 7-2, Sec. 21.4.4.1, for regions of high seismicity except that the required tie area is reduced to take into account the steel core. For a column where the steel core area is 4 percent of the total column area and the steel and concrete strengths are F_y = 50 ksi (344.5 Mpa) and f_c' = 6 ksi (41.3 MPa), respectively, Eq. 7.5.3.7.2 requires roughly 70 percent of the tie area required by Ref. 7-2 for a reinforced concrete column. The tie requirement of Eq. 7.5.3.7.2 is waived if the steel core of the composite member can alone resist the expected (arbitrary point in time) gravity load on the column. The rationale behind this is that additional confinement of the concrete is not necessary if the steel core can prevent collapse after an extreme seismic event. The load combination of $1.0D + 0.5L$ is based on a similar combination proposed by Ellingwood and Corotis (1991) as loading criteria for structural safety under fire conditions.

7.5.3.8 Additional Requirements in Special Moment Frames: Additional requirements for composite columns in C-SMF are based on similar requirements in Chapters 5 and 6 for steel and reinforced concrete columns in SMF. For additional commentaries, see Chapters 5 and 6 and the referenced sections of Ref. 7-2 and 7-3.

The strong-column/weak-beam philosophy follows that used for steel and reinforced concrete columns in SMF. For cases such as column bases where the formation of a plastic hinge in the column is inevitable, the detailing of the base should be such that the rotational ductility of the hinge is at least 6. For Seismic Performance Category E, special details such as steel jacketing of the column base should be considered to avoid spalling and crushing of the concrete.

Closed hoops are required to ensure that the concrete confinement and shear capacity are maintained under large inelastic deformations. The hoop detailing requirements are equivalent to those for reinforced concrete columns in SMF. The transverse reinforcement provisions are thought to be on the conservative side since composite columns generally will perform better than comparable reinforced concrete columns with similar confinement. However, without more research it is not clear to what degree the transverse reinforcement requirements can be reduced for composite columns. It should be recognized that the closed hoop and cross-tie requirements for C-SMF will require special details such as those suggested in Figure C7.5.3.8 to facilitate erection of the reinforcement around the steel core. Effective anchorage of ties must be assured to provide effective containment.

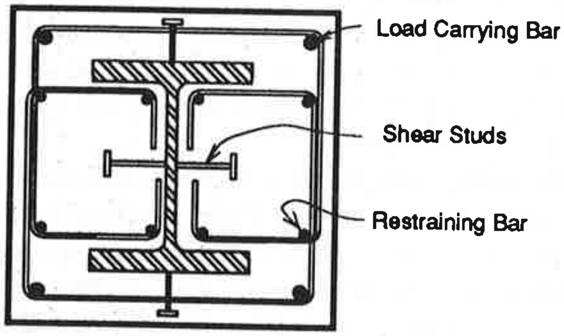


FIGURE C7.5.3.8 Example of a closed hoop detail for encased composite column.

7.5.4 FILLED COMPOSITE COLUMNS: The minimum ratio of the structural steel to column area of A_s/A_g equal to 4 percent follows that of Ref. 7-1.

7.5.4.1 Shear Strength: The shear strength of the filled member is conservatively limited to the shear yield capacity of the steel tube because there is no test evidence to demonstrate whether some portion of the concrete can be used for resisting shear. This approach is consistent with recommendations proposed by Furlong (1988) and the provisions in the latest draft of Eurocode 4 for composite construction (European Committee for Standardization, 1992). Even with this conservative approach, shear strength rarely governs the design of typical filled composite column sections with cross sectional dimensions up to 30 in. (762 mm). Alternate methods may be considered that treat the shear calculation for filled tubes in a similar manner to that for reinforced concrete columns, but where the steel tube is considered as shear reinforcement and its shear yielding strength is neglected. For example, using this approach the provisions of Chapter 11 of Ref. 7-2 could be used assuming $A_v/s = 2t$, where A_v and s are the area and spacing of shear reinforcement and t is the wall thickness of the tube. Given the upper limit on shear strength as a function of concrete crushing in Ref. 7-2, this approach would only be advantageous for columns with low ratios of structural steel to concrete areas.

7.5.4.2 Additional Requirements in Seismic Performance Categories D and E: See commentary to Sec. 7.5.3.7.

7.5.4.3 Additional Requirements in Special Moment Frames: The *b/t* and *D/t* ratios in Sec. 7.5.4.3 were determined based on the ratio of section slenderness criteria applied to members in steel OMF and SMF. Comparing the provisions in Ref. 7-1 and 7-3, the width/thickness ratios for SMF are about 0.8 of those for OMF. This same ratio of 0.8 was applied to the standard (nonseismic) *b/t* and *D/t* ratios for filled tubes in Ref. 7-1. The reduced slenderness criteria were imposed as a conservative measure until further research data becomes available on the cyclic response of filled tubes.

7.6 COMPOSITE CONNECTIONS: Due to the nature of composite construction and the pace of innovations, there are few standard details for connections in composite construction (Griffis, 1992; U.S.-Japan, 1992 and 1993). However, tests have been conducted for several types of connection details that demonstrate their potential for use in seismic design (see references cited previously in Sec. 7.4). In terms of seismic design, composite connections details often avoid or minimize the use of stiffeners and heavy welding compared to structural steel details. Also, compared to reinforced concrete, there are fewer instances where anchorage and development of primary beam reinforcing bars is a problem.

Given the many alternate configurations of composite structures and connections that are possible, there is no single comprehensive standard to draw upon for the design of connections. In most composite structures built to date, connections have been designed using basic mechanics, equilibrium, existing standards for steel and concrete construction, and research findings. The provisions in Sec. 7.6 are intended to establish basic behavioral assumptions for developing design models that satisfy equilibrium of internal forces in the joint for seismic design.

7.6.1 GENERAL REQUIREMENTS: The requirements for deformation capacity apply to both connections designed for gravity load only and connections that are part of the lateral-force-resisting system. The ductility requirement for gravity load only connections follows the recommendations of Sec. 8.2 of Ref. 7-3 and is intended to avoid brittle failure in gravity connections (e.g., Figure C7.6.1) that may have rotational restraint but limited rotation capacity.

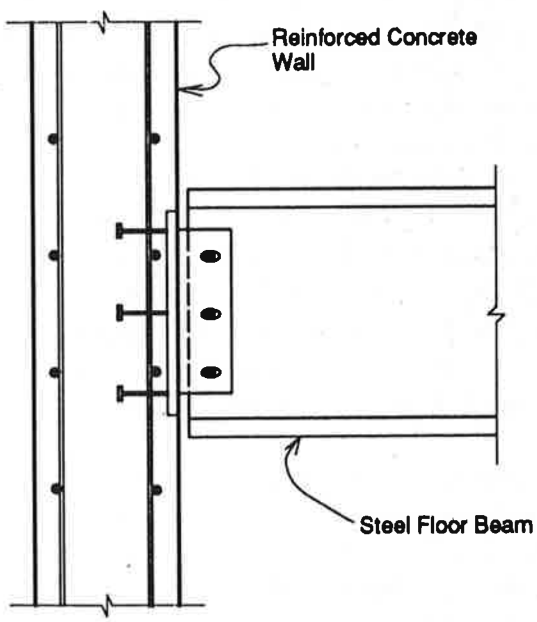


FIGURE C7.6.1 Steel beam-to-RC wall shear connection.

In calculating the required strength of connections based on the nominal strength of the connected members, allowance should be made for all components of the members that may increase their capacity above the nominal strength calculated in design. One example where this occurs is in beams where the additional capacity provided by reinforcement within an effective width of a concrete slab may be neglected in the strength calculation for a steel beam for positive moment capacity but should not be ignored when computing the negative moment capacity at a critical section such as the connection area. Another example that was cited previously is in concrete-filled tubular braces where the increased tension and compression strength of the brace due to concrete should be considered in determining the required connection strength.

7.6.2 STRENGTH DESIGN CRITERIA:

7.6.2.1 Force Transfer Between Structural Steel and Concrete: In general, forces between structural steel and concrete will be transferred by a combination of bond, adhesion, friction, and direct bearing. Transfer by bond and adhesion are not permitted for strength calculation purposes because: (a) these mechanisms are not effective in transferring load under inelastic load reversals and (b) the effectiveness of the transfer is highly variable depending on the surface conditions of the steel and shrinkage and consolidation of the concrete.

7.6.2.2 Structural Steel Elements: In many composite connections, steel components are encased by concrete that will inhibit or fully prevent local buckling. For seismic design where inelastic force reversals are likely, concrete encasement will be effective only if it is properly confined. One method of confinement is by reinforcing bars that are fully anchored into the confined core of the member (e.g., see requirements for hoops in Chapter 21 of Ref. 7-2). Adequate confinement also may occur without special reinforcement where the concrete cover is very thick. The effectiveness of the latter type of confinement should be substantiated by tests.

7.6.2.3 Shear and Bearing Stresses in Concrete: Transfer by friction shall be calculated using the shear friction provisions in Ref. 7-2 where the friction is provided by the clamping action of steel ties or studs or from compressive stresses under applied loads. Since the provisions for shear and friction in Ref. 7-2 are based largely on monotonic tests, the values are reduced by 25 percent where large inelastic stress reversals are expected. This reduction is a conservative measure that does not appear in Ref. 7-2, but is applied in these provisions for composite connections due to the relative lack of experience with certain configurations of composite construction.

7.6.2.4 Panel Zones: As shown in Figure C7.6.2.4, for fully encased steel connections, the total shear strength may be calculated as the sum of contributions from the reinforced concrete and steel shear panels. This method is supported by test data reported by Deierlein et al. (1989), Kanno (1993), and others. Further guidelines for calculating the shear strength of fully encased connections are presented by Deierlein et al. (1989) and ASCE (1994).

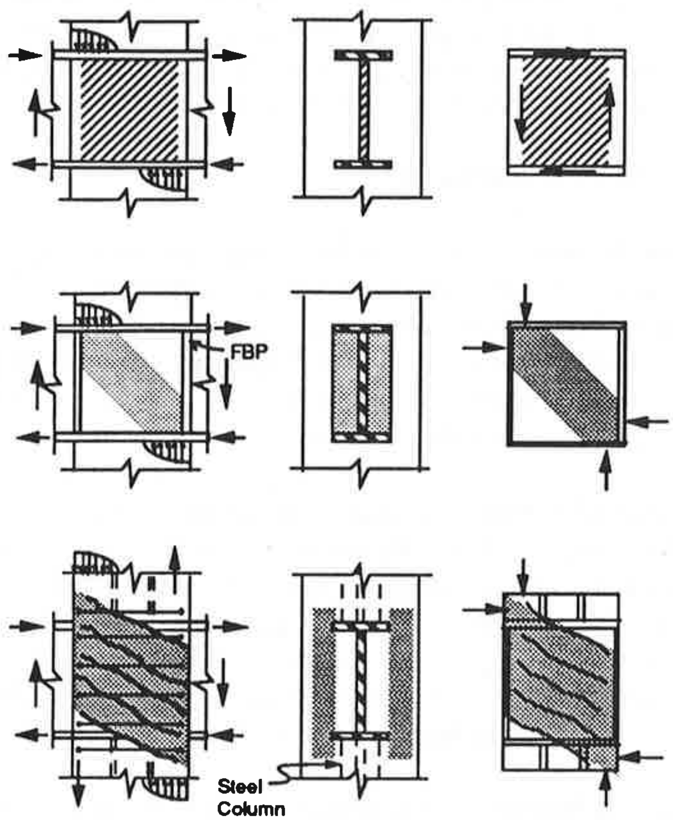


FIGURE C7.6.2.4 Panel shear mechanisms in steel beam-to-reinforced concrete column connections (Deierlein et al., 1989).

7.6.2.5 Reinforcing Bar Detailing Provisions: Reinforcing bars in and around the joint region serve the dual functions of resisting calculated internal tension forces and providing confinement to the concrete. Internal tension forces can be calculated using established engineering models that satisfy equilibrium (e.g., classical beam-column theory, the truss analogy, strut and tie models). Tie requirements for confinement usually are based on empirical models based on test data and past performance of structures (e.g., American Concrete Institute, 1991, Kitayama et al., 1987).

Slab Reinforcement in Connection Region: In some types of connections, the force transfer between the concrete slab and the steel column requires careful detailing. For composite PRF connections (Figure C7.4.1), the capacity of the concrete bearing against the column flange should be checked. Only the solid portion of the slab (area above ribs) should be counted, and the bearing strength should be limited to $1.2 f_c'$ (Ammerman et al., 1990). In addition, because the force transfer implies a large compressive strut forming between the slab bars and the column flange, adequate transverse steel shall be provided to form the tension tie. From equilibrium calculations, this amount should be the same as that provided as longitudinal reinforcement and should extend at least 12 in. (305 mm) on either side of the effective width strip.

Transverse Reinforcement in Columns Near Joint: Information on reinforcement requirements for details such as shown in Figure C7.4.2-1 is reported by Deierlein et al. (1989)

and Kanno (1993). Information based on experience with joints in reinforced concrete structures is reported by Sheikh (1980), Park et al. (1982), Saatcioglu (1991), and others.

Reinforcing Bar Development Lengths: Due to the limited size of joints, it often is impossible to provide the full development lengths specified in Ref. 7-2 for reinforced concrete members. Nevertheless, full development lengths should be provided unless test data are available to substantiate reduced development requirements. In any case, the development length shall not be less than that provided in American Concrete Institute (1991).

Longitudinal Reinforcement in Columns Near Joints: As in reinforced concrete joints, the large transfer of forces to column bars through the joint usually results in slippage of the bars under extreme loadings. Current practice for reinforced concrete joints is to control this slippage by limiting the maximum longitudinal bar sizes as described in ACI 352R-91.

REFERENCES:

American Concrete Institute. 1991. *Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures*, ACI 352R-91, 22 pp. Detroit, Michigan: ACI.

American Institute of Steel Construction. 1992. *Load and Resistance Factor Design of W-shapes Encased in Concrete*. Chicago, Illinois: AISC.

American Society of Civil Engineers. 1994 (in press). "Guidelines for Design of Joints Between Steel Beams and Reinforced Concrete Columns, STD in Steel and Concrete Task Committee on Design Criteria for Composite Structures," ASCE *Journal of Structural Engineering*.

Architectural Institute of Japan. 1987. *AIJ Standards for Structural Calculation of Steel Reinforced Concrete Structures*. Shiba, Minato-ku, Tokyo, Japan: Architectural Institute of Japan, 5-26-20.

Ammerman, D. J., and R. T. Leon. 1990. "Unbraced Frames with Semi-Rigid Connections," *AISC Engineering Journal* (April):12-21.

Azizinamini, A., and B. A. Prakash. 1993. "An Innovative Connection Detail for High Rise Buildings." In *Proceedings of the 1993 Structures Congress*, ASCE, April, pp. 1220-1225.

Azizinamini, A., and S. S. B. Kuska. 1993. Flexural Capacity and Ductility of High Strength Concrete Columns, paper by researchers at the University of Nebraska, submitted for publication.

Bing, L., R. Park and H. Tanaka. 1991. "Effect of Confinement on the Behavior of High Strength Concrete Columns Under Seismic Loading," paper presented at the Pacific Conference on Earthquake Engineering, New Zealand, November 1991.

Bjorhovde, R. 1984. "Effects of End Restraint on Column Strength - Practical Applications," *AISC Engineering Journal*, pp. 1-13.

Camacho, D. and T.V. Galambos. 1993. "Development of a Moment-Resisting One-Story Frame System for Seismic Resistance," work in progress sponsored by the Steel Joist Institute.

Chang, C.-M. and V. V. Bertero. 1986. *Earthquake Simulation Tests and Associated Studies of 0.3 - Scaled Model of Six-Story Concentrically Braced Steel Structure, Report No. UCB/EERC-86/10.* Berkeley, California: Earthquake Engineering Research Council.

Chrysostomou, C. Z. 1991. Effects of Degrading Infill Walls on the Nonlinear Seismic Response of Two-Dimensional Steel Frames, Ph.D. thesis, School of Civil and Environmental Engineering, Cornell University, Ithaca, New York.

Collins, M. P., D. Mitchell, and J. G. MacGregor. 1993. "Structural Design Consideration for High-Strength Concrete," ACI *Concrete International* (May):27-34.

Deierlein, G. G., T. M. Sheikh, J. A. Yura, and J. O. Jirsa. 1989. "Part 2: Beam Column Moment Connections for Composite Frames," ASCE *Journal of Structural Engineering* 115(11):2877-2896.

DiCorso, P. J., A. M. Reinhorn, J. R. Dickerson, J. B. Radziminski, and W. L. Harper. 1989. *Response of Frames with Bolted Semi-Rigid Connections - Part I - Experimental Study and Analytical Predictions*, Technical Report NCEER-89-0015. Buffalo, New York: National Center for Earthquake Engineering Research.

Easterling, W. S., and M. L. Porter. 1993. "Steel Deck Reinforced Concrete Diaphragms: Part I and Part II," accepted for publication in ASCE *Journal of Structural Engineering*.

Ellingwood, B. R., and R. B. Corotis. 1991. "Load Combinations for Buildings Exposed to Fires," *Engineering Journal* 28(1):37-44.

European Committee for Standardization. 1992. *Eurocode 4, Design of Composite Steel and Concrete Structures Part 1-1: General Rules and Rules for Buildings*, Final Draft. Brussels, Belgium: Europeon Committee for Standardization.

Furlong, R. W. 1988. "Steel-Concrete Composite Columns-II." In *Steel-Concrete Composite Structures*, edited Ed. R. Narayanan, Elsevier Applied Sciences, 211.

Fustok, M., S. C. Goel, and J. K. Wight. 1991. *Use of Steel Braces to Resist Seismic Forces in Precast Concrete Structures*, Research Report UMCE 91-13. Ann Arbor: The University of Michigan, Department of Civil Engineering.

Garcia, D. T., and A. H. Nilson. 1990. *A Comparative Study of Eccentrically-Loaded High-Strength Concrete Columns*, Structural Engineering Report 90-2. Ithaca, New York: Cornell University, School of Civil and Environmental Engineering.

Goel, S. C., and S. Lee. 1992. "A Fracture Criterion for Concrete-Filled Tubular Bracing Members Under Cyclic Loading." In *Proceedings*, ASCE Structures Congress, San Antonio, Texas, April 13-15, pp. 922-925.

Griffis, L. 1992. "Composite Frame Construction." In *Constructional Steel Design: An International Guide*, edited by P. J. Dowling, J. E. Harding, and R. Bjorhovde, pp. 523-553. Elsevier Science Publishers.

Harries, K., D. Mitchell, W. D. Cook, and R. G. Redwood. 1993. "Seismic Response of Steel Beams Coupling Concrete Walls," ASCE *Journal of Structural Engineering* 119(12):3611-3629.

Itani, A. M., and S. C. Goel. 1991. *Earthquake Resistance of Open Web Framing Systems*, Research Report UMCE 91-21. Ann Arbor: The University of Michigan, Department of Civil Engineering.

Kanno, R. 1993. Strength, Deformation, and Seismic Resistance of Joints Btween Steel Beams and Reinforced Concrete Columns, Ph.D. thesis, School of Civil and Environmental Engineering, Cornell University, Ithaca, New York.

Kitayama, K., S. Otani, and H. Aoyama. 1987. "Earthquake Resistant Design Criteria for Reinforced Concrete Interior Beam-Column Joints." In *Proceedings* - Pacific Conference on Earthquake Engineering, Wairakei, New Zealand, 1987.

Leon, R. T., and G. P. Forcier. 1992. "Parametric Study of Composite Frames." In *Connections in Steel Structures II*, Proceedings of the Second International Workshop on Connections in Steel Structures, edited by R. Bjorhovde et al., pp. 152-159. Chicago, Illinois: American Institute of Steel Construction.

Leon, R. T. 1990. "Semi-Rigid Composite Construction," *Journal of Constructional Steel Research* 15(2):99-120.

Leon, R. T., and D. J. Ammerman. 1990. "Semi-Rigid Connections for Gravity Loads," *AISC Engineering Journal* (April):3-11.

Liu and Chen. 1984. See p. 7-3.

Nader, M. N., and A. Astaneh. 1992. "Seismic Design Concepts for Semi-rigid Frames." In *Proceedings of ASCE Structures Congress' 92*, pp. 971-975. New York, New York: ASCE.

Naeim, Farzad, Ed. 1989. *Seismic Design Handbook*, Structural Engineering Series. New York: Van Nostrand Reinhold.

Nishiyama, I., T. Hasegawa, and H. Yamanouchi. 1990. *Strength and Deformation Capacity of Reinforced Concrete Column to Steel Beam Joint Panels*, Report 71, September, published by Building Research Institute, Ministry of Construction, Tsukuba, Japan (in Japanese).

Park, R., M. J. N. Priestley, and W. D. Gill. 1982. "Ductility of Square Confined Concrete Columns," ASCE *Journal of Structural Engineering* 108(ST4):929-950.

Ricles, J. M., and E. P. Popov. 1989. "Composite Action in Eccentrically Braced Frames," ASCE *Journal of Structural Engineering* 115(8):2046-2066.

Saatcioglu, M. 1991. "Deformability of Steel Columns." In *ACI 127.5*. Detroit, Michigan: American Concrete Institute.

Shahrooz, B. M., M. E. Remmetter, and F. Qin. 1993. "Seismic Design and Performance of Composite Coupled Walls," ASCE *Journal of Structural Engineering* 119(11):3291-3309.

Sheikh, S. A., and S. M. Uzumeri. 1980. "Strength and Ductility of Tied Columns," ASCE *Journal of Structural Engineering* 106(ST5):1079-1102.

Sheikh, T. M., G. G. Deierlein, J. A. Yura, and J. O. Jirsa. 1989. "Part 1: Beam Column Moment Connections for Composite Frames," ASCE *Journal of Structural Engineering* 115(11):2859-2876.

Steager, A., and R. T. Leon. 1993. *Design of Semi-Rigid Composite Frames*, Design Guide 10. Chicago, Illinois: American Institute of Steel Construction.

Steel Deck Institute. 1993. *Steel Deck Institute Design Manual*, Publication 27.

Steel Deck Institute. 1987. *Steel Deck Institute Diaphragm Design Manual*, Second Edition.

U.S.-Japan Planning Group. 1992. *Recommendations for U.S.-Japan Cooperative Research Program - Phase 5: Composite and Hybrid Structures*, Report UMCEE 92-29, edited by S. Goel. Ann Arbor: Department of Civil and Environmental Engineering, The University of Michigan.

U.S.-Japan Planning Group. 1993. *Proceedings of a U.S.-Japan Cooperative Research Program Workshop on Composite and Hybrid Structures*, Report UMCEE 93, edited by S. Goel. Ann Arbor: Department of Civil and Environmental Engineering, The University of Michigan.

Vulcraft. 1990. *Steel Floor and Roof Deck Manual*, Division of Nucor Corporation.

Zandonini, R. 1989. "Semi-Rigid Composite Joints." In *Structural Connections: Stability and Strength*, edited by R. Narayanan,. pp. 63-120. London: Elsevier Applied Science.

Zandonini, R., and Leon, R. T. 1992. "Composite Connections." In *Constructional Steel Design: An International Guide*, edited by P. J. Dowling, J. E. Harding and R. Bjorhovde, pp. 501-522. London: Elsevier Science Publishers.

Zaremba, C. J. 1988. "Strength of Steel Frames Using Partial Composite Girders," ASCE *Journal of Structural Engineering* 114(8):1741-1760.

Chapter 8 Commentary

MASONRY STRUCTURE DESIGN REQUIREMENTS

8.1 GENERAL:

8.1.1 SCOPE: The provisions of Chapter 8 govern design and construction of all types of masonry. Quality assurance is covered with a reference to Sec. 1.6. Reinforced and plain (unreinforced) masonry elements that are part of the basic structural system and those that are not part of the basic structural system are included.

8.1.2 REFERENCE DOCUMENTS: Design and construction standards cited in Chapter 8 are listed in Sec. 8.1.2. The materials standards are specifically listed to include only those materials permitted by the provisions.

The listing includes the document's designation, the year of the edition and the title of the document.

8.1.3 DEFINITIONS: Terms used in the provisions which have a specific meaning which differs from the dictionary definition are defined in Sec. 8.1.3. All other terms are defined by the dictionary.

8.1.4 NOTATIONS: Notations used in the provisions are defined in Sec. 8.1.4. English units of measure are stated followed by the metric unit in parenthesis for each term.

8.2 CONSTRUCTION REQUIREMENTS:

8.2.1 GENERAL: Ref. 8-2 is a standard specification prepared under consensus procedures. It was developed by members representing construction, design, materials, and research of masonry structures. The document is intended to be incorporated into contract documents used to construct masonry structures.

This standard specification was developed to be used in conjunction with *Building Code Requirements for Masonry Structures*, Ref. 8-1. Appropriate standards for materials and test methods are referenced. In addition to a general section, there are sections on masonry, reinforcement and metal accessories, and grout.

The materials listed in Ref. 8-2 have been restricted in order to obtain more predictable behavior and better performance required for strength design. Construction provisions found in Chapter 8 override those found in Ref. 8-2.

8.2.2 QUALITY ASSURANCE: See Sec. 1.6 of these provisions and commentary.

Quality assurance requirements for masonry structures include testing of masonry components (mortar, grout, and units) or testing of masonry assemblages. Industry guidelines for materials testing are listed below.

1. Brick Institute of America, 11490 Commerce Park Drive, Reston, Virginia 22091, *Technical Notes on Brick Construction*:

   No. 39 Revised, "Testing for Engineered Brick Masonry: Brick, Mortar and Grout," January 1987.

   No. 39A, "Testing for Engineered Brick Masonry: Determination of Allowable Design Stresses," December 1987.

   No. 39B, "Testing for Engineering Brick Masonry: Quality Assurance," March 1988.

2. National Concrete Masonry Association, 2302 Horse Pen Road, Herndon, Virginia 22071-3499:

   TEK 22A, *Prism Testing for Engineered Concrete Masonry*, 1979.

   TEK 107, *Laboratory and Field Testing of Mortar and Grout*, 1979.

   TEK 108, *Testing Concrete Masonry Assemblages*, 1979.

Industry guidelines for field inspection are listed below.

1. Brick Institute of America, 11490 Commerce Park Drive, Reston, Virginia 22091, *Technical Notes on Brick Construction*:

   No. 17C, "Reinforced Brick Masonry: Inspectors' Guide," May 1986.

2. National Concrete Masonry Association, 2302 Horse Pen Road, Herndon, Virginia 22071-3499:

   TEK 65, *Field Inspection of Engineered Concrete Masonry*, 1975.

   TEK 132, *Inspector's Guide for Concrete Masonry Construction*, 1983.

8.3 GENERAL DESIGN REQUIREMENTS:

8.3.1 SCOPE: This chapter offers three different methods for designing masonry structures. Any method, used within the limitations imposed, provides acceptable masonry construction with acceptable seismic resistance characteristics.

8.3.2 EMPIRICAL MASONRY DESIGN: Empirical design methods are based on the successful performance of masonry buildings. Prescriptive requirements and limited exposure to loads are necessary to ensure compliance.

The design process results in sizes and proportions of masonry elements using minimum thicknesses and maximum spans. Although rudimentary stress calculations are made, empirical masonry design does not require a complete structural analysis.

8.3.3 PLAIN (UNREINFORCED) MASONRY DESIGN: Design methods for plain masonry, often referred to as unreinforced masonry.

The procedures utilize working stress design provisions using principles of mechanics.

8.3.4 REINFORCED MASONRY DESIGN: Reinforcing steel complements the high compressive strength of masonry with high tensile strength. Increased load-carrying capacity and greater ductility result from the use of reinforcing steel.

8.3.5 SEISMIC PERFORMANCE CATEGORY A: Masonry structures designed in accordance with any one of the design methods listed perform adequately in Seismic Performance Category A. Therefore, no special requirements are needed.

8.3.6 - 8.3.9 SEISMIC PERFORMANCE CATEGORIES B THROUGH E: Based on an increasing need for ductility and strength as seismic resistant performance requirements increase, prescriptive requirements for reinforcement, masonry unit configurations, and materials are included for each of these Seismic Performance Categories. Observations of performance of masonry structures in earthquakes provide the basis for these requirements.

8.3.6 SEISMIC PERFORMANCE CATEGORY B: The use of empirical masonry design, Sec. 8.3.2, for the lateral load resisting system is not appropriate for Seismic Performance Category B. Masonry walls that are not part of the lateral load resisting system may be designed by the empirical method.

8.3.9 SEISMIC PERFORMANCE CATEGORY E:

8.3.9.2: To be effective, reinforcement in hollow unit masonry must be carefully located and held in position during grouting. Horizontal reinforcement may be held in position by tying to the securely held vertical bars or by means of tight fitting positioners.

8.3.10: PROPERTIES OF MATERIALS:

8.3.10.1: The given modulus of elasticity of steel reinforcement is taken from previous codes and is consistent with established design values. Design may be based on tested values of modulus of elasticity; however, these tests are rarely performed because it is impractical to test materials to be used in the construction at the time when the project is being designed.

8.3.10.2: Modulus of elasticity of masonry is used in determining stiffness of structural components prior to cracking. Therefore, the modulus is taken from the elastic portion of the stress strain curve. The modulus of elasticity of masonry is not clearly related to any property of mortar, unit, grout or prism h/t, but is influenced by all of these. TS5 concluded it was best to relate the value of E_m to the specified compressive strength of masonry. This is because f_m' is also influenced by these parameters. The 750 multiplier is used rather than lower multipliers reported (Wolde-Tinsae, 1993) since the actual compressive strength of masonry must exceed the specified compressive strength.

8.3.10.4: Research has been performed on structural masonry components having a compressive strength in the range of 1,500 to 6,000 psi (10 to 41 MPa). Design criteria are based on these research results. Design values therefore are limited to compressive strengths in the range of 1,500 to 4,000 psi (10 to 28 MPa) for concrete masonry and 1,500 to 6,000 psi (10 to 41 MPa) for clay masonry.

8.3.10.5: Modulus of rupture values in Table 8.3.10 are based on allowable working stress values for flexural tension multiplied by 2.0 to approximate the lower limit of strength values. See the Commentary to Ref. 8-1 for discussion. Stack bond masonry has historically been assumed to have no flexural bond strength across the head joints; thus, the grout area alone is used.

8.3.10.6: Research conducted on reinforced masonry components used Grade 60 reinforcement. To be consistent with laboratory documented performance, design is based on a steel yield strength that does not exceed 60,000 psi (413 MPa).

8.3.11 SECTION PROPERTIES: Section properties of masonry members are available in masonry design publications. Design is based on specified dimension. Actual dimensions may vary within the tolerance range given in the construction requirement (i.e., Ref. 8-2). The strength reduction factors are based in part on an anticipated variation in the specified (design) dimensions.

8.5 STRENGTH AND DEFORMATION REQUIREMENTS:

8.5.3 DESIGN STRENGTH: The design strength of a member and its connections is calculated by engineering principles and materials strength and yield values. This calculated strength is the nominal strength of the member. The nominal strength is less than the expected or mean strength because minimum guaranteed values or specified strengths are used for the calculations of nominal strength. A strength reduction factor, ϕ, is used to reduce the nominal strength to a design strength. The strength reduction factor, ϕ, is a variable that is dependent on the material and material behavior. Flexural strength of reinforced members is reduced less by the ϕ factor than is shear strength. Exceeding of the flexural strength of a reinforced member causes yielding of the reinforcement but not strength degradation. Exceeding of the shear strength results in a strength degradation.

Flexure Without Axial Load: The strength reduction factor for reinforced masonry is greater than for plain masonry because plain masonry after cracking lacks ductile performance.

Axial Load and Axial Load with Flexure: If the axial load results in balanced strain conditions (flexure produces strain in the reinforcement equal to the yield strain and strain in the masonry equal to the maximum usable strain, ϵ_{mu}) and the flexural reinforcement is minimal, an increase in flexural moment can cause compressive stresses in excess of the compressive strength. The failure will not be ductile; therefore, the strength reduction factor is more severe. Linear interpolation of the strength reduction factor is allowed since the required axial strength due to factored load, P_u, decreases from the axial load resulting in balanced strain conditions to zero, so as to make the transition linear from axial load with flexure to flexure without axial load.

The strength reduction factor for the vertical members of wall frames is more restrictive than for shear walls or coupled shear walls. The strength reduction factor for the vertical members of wall frames does not have a linear variation to its value. When $P_u/A_n f'_m$ is equal to 0.1, the strength reduction factor will be equal to 0.65.

The strength reduction factor for plain masonry members is unchanged from that factor that is applied for flexure only. Axial load increases the flexural capacity of plain masonry but does not significantly change its lack of ductility.

Shear: Strength reduction factors for calculation of design shear strength are commonly more severe than those factors used for calculation of design flexural strength. This concept is partially supported by the wider variance of shear capacities that have been obtained from experimental testing. The variance of the results of each experiment from the body of data is due not only to the variability of the masonry materials, the test apparatus and test methods, and the shear strength parameters tested but also to the greater sensitivity of shear resistance mechanisms to those factors.

Bearing: Exceeding of the bearing capacity causes crushing and spalling of bearing surfaces. The strength reduction factors given are those established for elements that have strength degradation.

8.5.4 DEFORMATION REQUIREMENTS: Stiffness of a structural element is as important or more important than strength. Stiffness is critical for serviceability and control of displacements. Drift of an element is the movement of one story of the building relative to the adjacent stories or the displacement of the shear wall relative to its fixed base. Drift of the top level of a shear wall is affected by foundation flexibility but the structural stresses and strains in the wall would not be increased by foundation flexibility.

The product of the effective moment of inertia, I, and the effective modulus of elasticity, E, is usually used as a variable for the calculation of the deformation of reinforced elements. The variability in I is caused by tensile cracking of the masonry cross section. If tensile cracking is not acceptable, as for plain masonry, I has a single value and the compressive modulus of elasticity and the moment of inertia of the gross cross section is used for the calculation of deformation.

If tensile cracking in anticipated, such as for reinforced masonry, the effective I at every cross section of the wall or beam is dependent on the curvature of the cross section and the shear deformation of each increment of the member length. Several nonlinear finite element programs have the capability of determining the stiffness degradation of reinforced masonry elements, but the effective stiffness, I, can be determined by use of Eq. 8.5.4.3.

The cracking moment is calculated using the section modulus of the gross section of wall times the modulus of rupture of masonry, f_r. The moment of inertia of the cracked section is calculated about the neutral axis of the section, using the masonry properties, and transforming the reinforcement into equivalent masonry areas by use of the ratio of the compressive modulus of steel and masonry. The cracked moment of inertia, I_{cr}, and the compressive modulus of masonry, E_m, is used to calculate the effective moment of inertia, I_{eff}.

Eq. 8.5.4.3 has been used as a means of providing a transition in stiffness between gross moment of inertia and a totally cracked section. Abboud (1987), Abboud and Hamid (1987), Abboud et al. (1990 and 1993), Hamid et al. (1989), and Horton and Tadros (1990) give additional insight and behavior for computing deflection for masonry components.

8.6 FLEXURE AND AXIAL LOADS:

8.6.2: The design principles listed are those that traditionally have been used for reinforced masonry members. The theory used for design of normally proportioned flexural members has limited applicability to deep flexural members. Shear warping of the cross section and a combination of diagonal tension stress and flexural tension stresses in the body of the deep flexural members require that deep beam theory be used for members that exceed the specified limits of span to depth ratio.

8.6.2.2: In the inelastic design of reinforced concrete beams, it is customary to limit the amount of tensile reinforcement to less than so-called "balanced reinforcement" -- that is, the quantity of reinforcement required to produce a condition in which the strain varies linearly from yield strain in the extreme fiber tension reinforcement, to the maximum usable strain (usually taken as 0.003) in the extreme compressive fiber of the concrete.

Because the actual yield strength of reinforcement normally exceeds the minimum specified value, and because of the possibility of strain hardening in the reinforcement, balanced reinforcement provisions like those discussed above still admit the possibility of extreme fiber strain exceeding the maximum usable strain in the compressive block. In addition, because they are not customarily enforced in the design of reinforced concrete columns, it is possible to have columns whose axial flexural capacity is controlled by the brittle failure mechanism of concrete crushing.

These characteristics could be serious shortcomings in applying the balanced reinforcement concept to the design of masonry flexural elements. Extensive research with many types of masonry flexural elements, carried on as part of the Technical Coordinating Committee for Masonry Research (TCCMAR) program, has shown that the flexural capacity of masonry elements degrades unacceptably under reversed cyclic drifts consistent with peak compressive strains exceeding the strain at peak stress, which has a value of about 0.002. It is therefore desirable to limit the amount of flexural reinforcement to that amount corresponding to a critical condition in which all or most of the tension reinforcement is yielded, and the extreme fiber compressive strain has just reached 0.002. To calculate that amount of reinforcement, it is necessary to estimate the stress variation in the tensile reinforcement and in the compressive stress block.

When flexural cracking occurs on the tension side of a flexural element, a discontinuous strain field is created. At average strains less than the strain hardening value, the strains in tensile reinforcement at the cracks greatly exceed the strain hardening value, and the tensile reinforcement begins to strain harden at the cracks. As a consequence, stresses in tensile reinforcement should be computed assuming a steel stress greater than yield, especially in view of the typical overstrength of reinforcing steel compared to the minimum specified value. Because of these factors, it is felt prudent to compute the maximum force from the tensile reinforcement in the above critical condition using an average stress of $1.25f_y$.

In that critical condition, when the extreme fiber compressive strain equals 0.002, the stress-strain behavior of the compressive reinforcement and the masonry is essentially linear and elastic. That is, stresses can be computed from strains using the appropriate material modulus.

Finally, to compute the amount of tensile reinforcement corresponding to the above critical condition, it is necessary to include the effect of axial loads. Increases in axial load and increases in tensile reinforcement affect the compressive stress block identically. As the compressive axial load on the element increases, the amount of tensile reinforcement necessary to produce critical conditions decreases. Under heavy compressive axial loads, the maximum amount of tensile reinforcement permitted to produce compliance with the above critical conditions may be less than that required to comply with minimum reinforcement requirements, or to resist out-of-plane loads. In such cases, it will be necessary to increase the depth of the wall section, reducing the axial load stress on the compressive stress block.

For further discussion, see He and Priestley (1992), Leiva and Klingner (1991), Limin and Priestley (1988), Merryman et al. (1989), Seible et al. (1992), and Shing et al. (1991).

8.6.3.5: The axial load strengths given by Eq. 8.6.3.5-1 and 8.6.3.5-2 are based on analysis of the results of axial load tests performed on clay and concrete masonry elements. For members having an h/r ratio not exceeding 99, the specimens failed at loads less than the Euler buckling load. Eq. 8.6.3.5-1 was empirically fit to test data for these members. For h/r values in excess of 99, the limited test data is adequately approximated by the Euler buckling equation.

8.7.3 DESIGN OF REINFORCED MASONRY MEMBERS: The development of strength design procedures for masonry requires a reasonably simplified and accurate equation that is capable of predicting the ultimate shear strength of a masonry wall. Once agreed upon, this equation, together with appropriate ϕ factors, will form a key part of strength design procedures.

Over the past two decades many hundreds of tests have been performed in the U.S., Japan and New Zealand to determine the strength and ductility of concrete block and clay brick shear walls subjected to cyclic lateral load patterns. From these tests come equations to predict the shear strength of walls usually calibrated to the tests carried out by the particular researcher. Fattal and Todd (1991) compared the predictions of four different equations with available experimental results. The only flaw in this work was that they included the *UBC* design equations with the inference that the *UBC* equations were predictive equations for the ultimate shear strength of masonry. This is not the intent of the *UBC* equations. They were developed and then modified as part of the code development process to provide a lower bound on the shear capacity of masonry walls.

Two other reports/papers were reviewed as part of preparing this overview document; Blondet et al. (1989) and Anderson and Priestley (1992) also looked at predictive equations which were more simplified than those included in the Fattal and Todd review. As a consequence, a total of six different predictive methods have been reviewed.

In summary, the methods include two or more of the following components:

$$V_u = V_m + V_{sh} + V_{sv} + V_p \qquad \text{(C8.7.3-1)}$$

where:

V_m = contribution of the masonry

V_{sh} = contribution of the horizontal steel

V_{sv} = contribution of the vertical steel

V_p = contribution of the axial load

The report by Fattal and Todd (1991) is quite thorough and the test data used to assess the Shing, Matsamura, and Architectural Institute of Japan (AIJ) predictive equations were also used to assess the methods proposed by Blondet et al. (1989) and Anderson and Priestley (1992) and the final TCCMAR equations that were developed as part of the TCCMAR study. The form of these equations are given in Table C8.7.3-2.

Rather than present the details of each of the test results that were developed, a statistical summary is provided in Table C8.7.3-1. This provides the overall average, standard deviation and coefficient of variation for all 62 tests included in the Fattal and Todd report. The values given in Table C8.7.3-1 are the ratio of the shear strength obtained by the predictive equation divided by the ultimate strength obtained from the test. A perfect prediction has a ratio of 1 and a conservative prediction has a ratio less than 1.

Also included in Table C8.7.3-1 are the mean values of the four different sets of tests. Test 1-10 are from Shing et al. (1991), Tests 11-28 are from Matsamura (1987), Tests 29-37 are from Okamoto et al. (1987), and Tests 38-62 are from Sveinsson et al. (1985).

As part of the TCCMAR studies, it was decided to use a combination of the Blondet et al. and Anderson and Priestley equations. In comparing the manner in which the two methods account for contribution of the masonry component, it was decided to use the Blondet form. As part of the Berkeley tests (Mayes et al., 1976, Chen et al., 1978, Hidalgo et al., (1978, 1979), it was concluded that the *M/Vd* ratio should be part of the masonry equation rather than just a straight function of 2.9 $\sqrt{f_m'}$ as in the Anderson and Priestley equation. Furthermore, there was very little numerical difference in the values used to account for the vertical load contribution. As a consequence, it was decided to use the more simplified form of $0.25\sigma_c$ used by Anderson and Priestley. The final form of the TCCMAR equation was given as:

$$v = (4 - 1.75M/Vd)\sqrt{f_m'} + 0.5\rho_h f_{yh} + 0.25\sigma_c \qquad \text{(C8.7.3-2)}$$

The metric equivalent of Eq. C8.7.3-2 is:

$$v = 0.083(4 - 1.75M/Vd)\sqrt{f_m'} + 0.5\rho_h f_{yh} + 0.25\sigma_c$$

Some members of TCCMAR believed that some contribution of vertical steel should be included and this issue was investigated. Many of the test specimens only included jamb steel and consequently two different vertical steel contributions were investigated: $1/4\rho_v f_{yv}$ and $1/4\rho_{vi} f_{yvi}$ where ρ_v is the total vertical steel and ρ_{vi} is only the interior vertical steel and neglects the jamb steel. The correlation and the test results were not as good when a contribution from vertical steel was included and consequently it was decided not to include it in the recommended TCCMAR shear equation.

Application of the shear strength equation to partially grouted masonry was based in part on Fattal (1993a and 1993b).

TABLE C8.7.3-1

Tests	Shing	Okamoto	Matsamura	Blondet et al.	Anderson & Priestley	TCCMAR
All 62 tests						
Mean	0.83	0.81	0.91	1.03	1.06	1.02
Standard Deviation	0.23	0.27	0.20	0.24	0.23	0.24
Coefficient of Variation	0.05	0.07	0.04	0.06	0.05	0.05
Mean Values						
Tests 1-10 (Shing)	0.94	1.25	0.93	0.88	1.02	0.87
Tests 11-27 (Matsumura)	0.89	0.82	0.99	1.10	1.13	1.07
Tests 27-38 (Okamoto)	0.65	0.76	0.75	0.80	0.86	0.81
Tests 39-62 (Sveinsson)	0.82	0.66	0.91	1.13	1.11	1.12

TABLE C8-7.3-2

	Masonry Component	Horizontal Steel	Vertical Load
TCCMAR	$\left(\frac{4 - 1.75M}{V_d}\right)\sqrt{f'_m}$	$0.5\rho_h f_h$	$0.25\sigma_c$
Blondet et al.	$\left(\frac{4 - 1.75M}{V_d}\right)\sqrt{f'_m}$	$0.5\rho_h f_h$	$\sqrt{V_m^2 + \frac{V_m \sigma_c}{1.5}} - V_m$
Anderson	$2.9\sqrt{f'_m}$	$0.5\rho_h f_h$	$0.25\sigma_c$
Shing[a]	$(0.0217\rho_v f_v + 0.166)\sqrt{f'_m}$	$\left(L - \frac{2d^1}{S_h} - 1\right)\frac{S_h}{L}\rho_h f_h$	$(0.0217\sigma_o)\sqrt{f'_m}$

	Masonry Component	Horizontal Steel	Vertical Load
TCCMAR	$\left(\frac{4-1.75M}{V_d}\right)\sqrt{f'_m}$	$0.5\rho_h f_h$	$0.25\sigma_c$
Matsamura[a]	$\left[\left(\frac{0.76}{r_d+0.7}+0.012\right)\left(4.04\rho_v^{0.3}\right)\sqrt{f'_m}\right]\frac{d}{L}$	$\left[0.01575(\rho_h f_h)^{1/2}\sqrt{f'_m}\frac{\delta d}{L}\right]$	$(0.175\sigma_o)\frac{d}{L}$
AIJ	$\left[4.64\rho_v^{0.23}\left(0.01f'_m+0.176\right)\left(\frac{1}{R_c+0.12}\right)\right]\frac{a}{l}$	$\left[0.739(\rho_N f_h)^{1/2}+0.739(\rho_V f_v)^{1/2}\right]\frac{d}{L}$	$(0.0875\sigma_o)\frac{d}{L}$

[a] These equations are in metric units.

8.8 SPECIAL REQUIREMENTS FOR MASONRY BEAMS:

8.8.1: Masonry beams may be loaded normal to their plane by wind or earthquake forces. The beam must have adequate strength to span between support points under the action of the out-of-plane loads. The arbitrary limits of 50 and 32 were judged to be adequate absolute limits on the unbraced span to beam width ratios for the conditions listed.

8.8.2: Gravity loading of a masonry beam may be applied eccentrically to its vertical centroidal plane. The lateral supports of the masonry building should restrain the beam from rotation under the eccentric action of the gravity load.

If the beam is supported laterally at one edge only (top or bottom), then the lateral support should have the moment capacity to restrain the rotation caused by loading normal to the face of the beam that is eccentric to the support point.

8.8.3: A minimum amount of flexural reinforcement in the positive moment zone of the beam is specified. This minimum is specified as a ratio, ρ, of the quantity of the reinforcement to the cross-sectional area of the beam. The minimum ratio specified is intended to require that the post-cracked moment capacity exceeds the uncracked moment capacity of the section.

These requirements for a minimum quantity of positive moment reinforcement assumes that cracking has occurred in zones of negative moment and that the change in beam stiffness has increased the positive moment. However, if the positive moment capacity of the reinforced section exceeds the uncracked positive moment capacity, transfer of moment to this zone is accommodated.

If a section of the adjacent concrete floor serves as the compression flange of the beam, minimum reinforcement is based on the masonry section which is in tension due to positive moment.

8.8.4 DEEP FLEXURAL MEMBERS: The theory used for design of beams has a limited applicability to deep beams. Shear warping of the cross section and a combination of diagonal tension stress and flexural tension stress in the body of the deep beam requires that deep beam theory be used for design of members that exceed the specified limits of span to depth ratio. Analysis of wall sections that are used as beams generally will result in a distribution of tensile stress that requires the lower one-half of the beam section to have uniformly distributed reinforcement. The uniform distribution of reinforcement resists tensile stress caused by shear as well as flexural moment.

The flexural reinforcement for deep beams must meet or exceed the minimum flexural reinforcement ratio of Sec. 8.8.3. Additionally, horizontal and vertical reinforcement must be distributed throughout the length and depth of deep beams and must provide reinforcement ratios of at least 0.001. Distributed flexural reinforcement may be included in the calculations of the minimum distributed reinforcement ratios.

Flexural reinforcement that is lumped entirely at the bottom and/or top of a deep flexural member, however, should be ignored when calculating the distributed horizontal reinforcement ratio. In such a case, the lumped flexural steel must provide a minimum flexural reinforcement ratio of $120/f_y$ in accordance with Sec. 8.8.3. For Grade 60 steel, this requirement is equivalent to a minimum flexural reinforcement ratio of 0.002.

Although this flexural reinforcement ratio results in twice the ratio required by Sec. 8.8.4.3, the flexural steel is lumped at the top and/or bottom of the beam and is not uniformly distributed. Since the intent of Sec. 8.8.4.3 is to ensure a minimum quantity of uniformly distributed reinforcement throughout the depth of the deep beam, the lumped flexural steel is not considered when calculating the minimum distributed reinforcement ratios.

8.9 SPECIAL REQUIREMENTS FOR COLUMNS:

8.9.1: Maximum and minimum limitations on the area of longitudinal reinforcement for columns are traditional values that have been in codes for many years. Minimum areas are limited so that creep of the masonry, which tends to transfer load from masonry to reinforcing steel will not result in increasing the stress in the steel to yield level. The maximum area limitation represents a practical limit on the amount of reinforcing steel in terms of economy and steel placement. No testing or research has been done to justify changes in these traditional values.

8.9.2: The minimum number of bars in columns also is a traditional number. It is obviously appropriate, however, to suit rectangular or square column shapes and tying requirements.

8.9.3: The lateral tie restrictions in this section are also traditional. The column tie bending requirements of Part c are to be as shown.

Reinforcement is restricted to an amount below the area required for flexural bending only in order to preserve a ductile failure condition (i.e., steel will reach ultimate yield strain before concrete reaches ultimate yield strain which would be defined as a brittle failure). It is therefore important to keep the reinforcement ratio low.

8.10 SPECIAL REQUIREMENTS FOR WALLS:

8.10.1: The flexural strength of reinforced walls loaded normal to the surface is required to exceed the uncracked flexural strength. The basis for this requirement is that a static over load on the wall may cause very large displacements before strain hardening in the reinforcement increases the cracked flexural strength to the value of the uncracked flexural strength.

8.11 SPECIAL REQUIREMENTS FOR SHEAR WALLS:

8.11.2 CONFINEMENT OF COMPRESSIVE STRESS ZONE: The compressive stress zone is required to be confined when the allowable ratio of intensity and/or overall drift of a wall frame is increased to 0.015. The allowable drift ratio assumes that the strain in the compression zone will exceed that considered acceptable for nonconfined compression zones.

8.11.3 FLANGED SHEAR WALLS: Tests on flanged shear walls (Priestley and Limin, 1990; Sieble et al., 1992) have indicated that if the conditions of Sec. 8.11.3.1 are satisfied, the flange will act in conjunction with the web as a part of the flexural member.

The tributary flange widths defined in Sec. 8.11.3.3 and 8.11.3.4 are considered to be values appropriate for predicting flexural behavior and strength. The values were taken from experimental results. This has significance when calculating probable shear force on the wall, which is related to the probable maximum flexural strength. For the calculation of maximum allowable reinforcement ratios, the reinforcement in the flange of the width specified in Sec. 8.11.3.4 must be considered as part of the maximum reinforcement ratio.

8.11.4 COUPLED SHEAR WALLS: Coupled shear walls are defined as shear walls in a common wall plane that are interconnected or coupled by spandrel beams. These beams are typically at each floor level. The coupling beams can be a section of a reinforced concrete floor that has continuity with the shear walls. Caution should be exercised to distinguish between coupled shear walls and walls with openings. In a coupled wall system, the yield limit state is allowed only in the coupling beam and at the base of the shear wall. If the flexure or shear yield state occurs in the wall between coupling beams, the system is a wall with openings. This system has very limited ductility and should be redesigned to prevent yielding in the reinforced wall at points other than the base of the shear wall.

Conformance with the requirement that the coupling beams reach their moment limit state at or before the shear wall reaches its moment limit state need not be checked if the ratio of the depth of the shear wall to the depth of the coupling beams exceeds 3 or more and the length of the coupling beams is less than one-half of the story height. Linear elastic analyses of the coupled wall system are inadequate to determine the yield status of the shear wall and the coupling beams. The stiffness of the shear wall will degrade rapidly in the first story. The shear walls in the upper stories may be uncracked.

8.11.4.2 Shear Strength of Coupling Beams: The nominal shear strength of coupling beams must be equal to the shear caused by development of a full yield hinge at each end of the coupling beams. This nominal shear strength is estimated by dividing the sum of the calculated yield moment capacity of each end of the coupling beams, M_1 and M_2, by the clear span length, L.

A coupling beam may consist of a masonry beam and a part of the reinforced concrete floor system. Reinforcement in the floor system parallel to the coupling beam should be considered as a part of the coupling beam reinforcement. The limit of the minimum width of floor that should be used is six times the floor slab thickness. This quantity of reinforcement may exceed the limits of Sec. 8.6.2.2 but should be used for the computation of the normal shear strength.

REFERENCES

Abboud, B. E., X. Lu, and F. C. Schmitt. 1993. "An Evaluation of Three Current Deflection Methods for Predicting the Lateral Deflection of Masonry Walls." In *Proceedings of the Sixth North American Masonry Conference*, Philadelphia, Pennsylvania, pp.73-85.

Abboud, B. E., and A. A. Hamid. 1987. "A Comparative Study of the Flexural Behavior of Reinforced Block Masonry and Reinforced Concrete Using Small Scale Model Walls." In *Proceedings of the Fourth North American Masonry Conference*, Los Angeles, California.

Abboud, B. E., A. A. Hamid, and H. G. Harris. 1990. "Small Scale Modeling of Concrete Block Masonry Structures," *ACI Structural Journal* 87(2):145-155.

Abboud, B. E. 1987. The Use of Small Scale Direct Models for Concrete Block Masonry Assemblages and Slander Reinforced Walls Under Out-of-Plane Loads, Doctoral Thesis, Drexel University, Philadelphia, Pennsylvania.

Blondet, J. M., R. L. Mayes, T. E. Kelley, R. R. Villablanca, and R. E. Klinger. 1989. *Performance of Engineered Masonry in the Chilean Earthquake of March 3, 1985: Implications for U.S. Design Practice*. Austin: Phil Ferguson Structural Engineering Laboratory, University of Texas.

Chen, S. W., P. A. Hidalgo, R. L. Mayes, R. W. Clough, and H. D. McNiven. 1978. *Cyclic Loading Tests of Masonry Single Piers*, Vol. 2, EERC Report UCB/EERC-78/27. Berkeley: University of California Earthquake Engineering Research Center.

Fattal, S. G. 1993a. *Strength of Partially Grouted Masonry Shear Walls*, Report NIST 5147. Gaithersburg, Maryland: National Institute of Standards and Technology.

Fattal, S. G. 1993b. *The Effect of Critical Parameters on the Behavior of Partially Grouted Masonry Shear Walls Under Lateral Loads*, NIST Report NISTIR 5116. Gaithersburg, Maryland: National Institute of Standards and Technology.

Fattal, S. G., and D. R. Todd. 1991. *Ultimate Strength of Masonry Walls: Prediction vs. Test Results*, NIST Report NISTIR 4633. Gaithersburg, Maryland: National Institute of Standards and Technology.

Hamid, A. A., M. K. Hatem, H. G. Harris, and B. E. Abboud. 1990. Ilysteretic Response and Ductility of Reinforced Concrete Masonry Walls Under Out-of-Plane Loading." In *Proceedings of the Fifth North American Conference*, Urbana, Illinois, pp. 397-410.

Hamid, A. A., B. E. Abboud, M. W. Farah, M. K. Hatem, and H. G. Harris. 1989. "Response of Reinforced Block Masonry Walls to Out-of-Plane Static Loads," in *U.S.-Japan Coordinated Program For Masonry Building Research Report 3.2(a)*, Drexel University.

He, L., and N. Priestley. 1992. *Seismic Behavior of Flanged Masonry Shear Walls - Final Report*, TCCMAR Report 4.1-2.

Hidalgo, P. A., R. L. Mayes, H. D. McNiven, and R. W. Clough. 1978. *Cyclic Loading Tests of Masonry Single Piers*, Vol. 1 and 3, EERC Report UCB/EERC-78/27 and 79/12. Berkeley, University of California Earthquake Engineering Research Center.

Horton, R. T., and M. K. Tadros. 1990. "Deflection of Reinforced Masonry Members," *ACI Structural Journal* 87(4):73-85.

Leiva, G., and R. Klingner. 1991. *In-Plane Seismic Resistance of Two-Story Concrete Masonry Shear Walls with Openings*, TCCMAR Report 3.1(c)-2.

Limin, H., and N. Priestley. 1988. *Seismic Behavior of Flanged Masonry Shear Walls*, TCCMAR Report 4.1-1.

Matsumura, A. 1987. "Shear Strength of Reinforced Hollow Unit Masonry Walls." In *Proceedings* of the 4th North American Masonry Conference, Los Angeles, California.

Mayes, R. L., Y. Omote, and R. W. Clough. 1976. *Cyclic Shear Tests of Masonry Piers*, Vol. 1, Report 76-8. Berkeley: University of California Earthquake Engineering Research Center.

Merryman, K., G. Leiva, N. Antrobus, and R. Klingner. 1989. *In-Plane Seismic Resistance of two-Story Concrete Masonry Coupled Shear Walls*, TCCMAR Report 3.1(c)-1.

Okamoto, S., Y. Yamazaki, T. Kaminosono, M. Teshigawara, and H. Hirashi. 1987. "Seismic Capacity of Reinforced Masonry Walls and Beams in Wind and Seismic Effects." In *Prooceedings* of the 18th Joint Meeting, U.S. - Japan Panel on Wind and Seismic Effects, ed. by N. J. Raufaste, Report NBSIR 87-3540. Gaithersburg, Maryland: National Institute of Standards and Technology.

Priestley, N., and H. Limin. 1990. "Seismic Response of T-Section Masonry Shear Walls." *TMS Journal* 9(1):10-19.

Seible, F., G. A. Hegemier, M. J. N. Priestley, G. R. Kingsley, A. Kurkchubasche, and A. Igarashi. 1992. *The U.S. - TCCMAR Five-Story Full Scale Masonry Research Building Test - Preliminary Report*, TCCMAR Report 9.2-4.

Shing, P. B., J. Noland, H. Spaeh, E. Klamerus, and M. Schuller. 1991. *Response of Single-Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads*, TCCMAR Report 3.1(a)-2.

Shing, P. B., M. Schuller, and V. S. Hoskere. 1990. "In-Plane Resistance of Masonry Shear Walls," *ASCE Journal of Structural Engineering* 116(3).

Sveinsson, B. I., H. D. McNiven, and H. Sucuoglu. 1985. *Cyclic Shear Tests of Masonry Piers*, Vol 4, Report UCB/EERC 85-15. Berkeley: University of California Earthquake Engineering Research Center.

Wolde-Tinsae, A. M., R. H. Atkinson, and A. A. Hamid. 1993. "State-of-the-Art Modulus of Elasticity of Masonry." In *Proceedings of the Sixth North American Masonry Conference*. Boulder, Colorado: The Masonry Society.

The following Technical Coordinating Committee for Masonry Research task reports not specifically cited providing the substantiating data for the strength design criteria presented in this chapter are available through the Earthquake Engineering Research Center Library in Richmond, California (phone 415-231-9403):

Task No. Author(s) and Title

1.1-1: Atkinson and Kingsley, *Comparison of the Behavior of Clay & Concrete Masonry in Compression*, September 1985. 151 pgs.

1.2(a)-1: Hamid, A. A., G. F. Assis, and H. G. Harris, *Material Models for Grouted Block Masonry*, August 1988. 67 pgs.

1.2(a)-2: Assis, G. F., A. A. Hamid, and H. G. Harris, *Material Models for Grouted Block Masonry*, August 1989. 134 pgs.

1.2(b)-1: Young, J. M., and R. H. Brown, *Compressive Stress Distribution of Grouted Hollow Clay Masonry Under Strain Gradient*, May 1988, 170 pgs.

1.3-1: Atkinson, R. H., *An Assessment of Current Material Test Standards for Masonry Limit States Design Methods*, June 1991. 38 pgs.

2.1-1: Hart, G., and M. Basharkhah, *Slender Wall Structural Engineering Analysis Computer Program (Shwall, Version 1.01)*, September 1987. 68 pgs.

2.1-2: Hart, G., and M. Basharkhah, *Shear Wall Structural Engineering Analysis Computer Program (Shwall, Version 1.01)*. September 1987. 75 pgs.

2.1-3: Nakaki, D., and G. Hart, *Uplifting Response of Structures Subjected to Earthquake Motions*, August 1987. 200 pgs.

2.1-4: Hart, G., N. Sajjad, and M. Basharkhah, *Inelastic Column Analysis Computer Program (INCAP, Version 1.01)*, March 1988.

2.1-5: Hong, W. K., G. C. Hart, and R. E. Englekirk, *Force-Deflection Evaluation and Models for University of Colorado Flexural Walls*, December 1989.

2.1-6: Hart, G. C., J. W. Jaw, and Y. K. Low, *SCM Model for University of Colorado Flexural Walls*, December 1989. 31 pgs.

2.1-7: Hart, G. C., N. Sajjad, and M. Basharkhah, *Inelastic Masonry Flexural Shear Wall Analysis Computer Program*, February 1990. 41 pgs.

2.1-8: Hart, G. C., R. Englekirk, M. Srinivasan, S. C. Huang, and D. J. Drag, *Seismic Performance Study, DPC Gymnasium, Elastic Time History Analysis Using SAP90*, February 1992. 41 pgs.

2.1-9: Hart, G. C., R. Englekirk, M. Srinivasan, S. C. Huang, and D. J. Drag, *Seismic Performance Study, TMS Shopping Center, Elastic Time History Analysis Using SAP90*, February 1992. 42 pgs.

2.1-10: Hart, G. C., R. Englekirk, J. W. Jaw, M. Srinivasan, S. C. Huang, and D. J. Drag, *Seismic Performance Study, RCJ Hotel*, February 1992. 51 pgs.

2.1-11: Hart, G. C., R. Englekirk, M. Srinivasan, S. C. Huang, and D. J. Drag, *Performance Study, 2-Story Masonry Wall-Frame Building*, February 1992. 112 pgs.

2.1-12: Hart, G. C., R. Englekirk, J. W. Jaw, M. Srinivasan, S. C. Huang, and J. D. Drag, *Seismic Performance Study, Designed by Tentative Limit Sates Design Standard*, February 1992. 75 pgs.

2.2-1: Ewing, R. D., A. El-Mustapha, and J. Kariotis, *FEM/I - A Finite Element Computer Program for the Nonlinear Static Analysis of Reinforced Masonry Building Components*, December 1987 (Revised June 1990). 124 pgs.

2.2-2: Ewing, R. D., *Parametric Studies on Reinforced Masonry Shear Walls Using FEM/I, A Nonlinear Finite Element Analysis Program*, March 1992.

2.2-3: Ewing, R. D., *Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard*, March 1992. 48 pgs.

2.3-1: Ewing, R., J. Kariotis, and A. El-Mustapha, *LPM/I, A Computer Program for the Nonlinear, Dynamic Analysis of Lumped Parameter Models*, August, 1987. 200 pgs.

2.3-2: Kariotis, J., A. El-Mustapha, and R. Ewing, *Influence of Foundation Model on the Uplifting of Structures*, July 1988. 50 pgs.

2.3-3: Kariotis, J., A. Rahman, and A. El-Mustapha, *Investigation of Current Seismic Design Provisions for Reinforced Masonry Shear Walls*, January 1990. 48 pgs.

2.3-4: Kariotis, J., A. Rahman, O. Waqfi, and R. Ewing, *Version 1.03 LPM/I - A Computer Program for the Nonlinear, Dynamic Analysis of Lumped Parameter Models*, February 1992. 227 pgs.

2.3-5: Kariotis, J., O. Waqfi, and R. Ewing, R., *A Computer Program Using Beam Elements for the Nonlinear, Dynamic Analysis of Lumped Parameter Models*, February 1992. 96 pgs.

2.3-6: Kariotis, J., and O. Waqfi, *Comparison of the Dynamic Response of a Damped MDOF Nonlinear Beam Model With an Equivalent SDOF Hysteretic Model*, April 1992. 88 pgs.

2.3-7: Kariotis, J., and O. Waqfi, *Recommended Procedure for Calculation of the Balanced Reinforcement Ratio*, February 1992. 73 pgs.

2.4(b)-1: Button, M. R., and R. L. Mayes, *Out-of-Plane Seismic Response of Reinforced Masonry Walls: Correlation of Full-Scale Test and Analytical Model Results*, March 1991. 65 pgs.

3.1(a)-1: Scrivener, J., *Summary of Findings of Cyclic Tests on Masonry Piers*, June 1986. 7 pgs.

3.1(a)-2: Shing, P. B., J. Noland, H. Spaeh, E. Klamerus, and M. Schuller, *Response of Single-Story Reinforced Masonry Shear Walls to In-Plane Lateral Loads*, January 1991. 136 pgs.

3.1(b)-1: Seible, F., and H. LaRovere, *Summary of Pseudo Dynamic Testing*, February 1987. 46 pgs.

3.1(b)-2: Igarashi, A., F. Seible, and G. Hegemier, *Development of the Generated Sequential Displacement Procedure and the Simulated Seismic Testing of the TCCMAR Three-Story In-Plane Walls*, June 1993.

3.1(c)-1: Merryman, K., G. Leiva, B. Antrobus, and R. Klingner, *In-Plane Seismic Resistance of Two-Story Concrete Masonry Coupled Shear Walls*, September 1989. 176 pgs.

3.1(c)-2: Leiva, G., and R. Klingner, *In-plane Seismic Resistance of Two-Story Concrete Masonry Shear Walls with Openings*, August 1991. 326 pgs.

3.2(a)-1: Hamid, A., B. Abboud, M. Farah, K. Hatem, and H. Harris, *Response of Reinforced Block Masonry Walls to Out-of-Plane Static Loads*; September 1989. 120 pgs.

3.2(b)-1: Agbabian, M., S. Adham, S. Masri, V. Avanessian, and V. Traina, *Out-of-Plane Dynamic Testing of Concrete Masonry Walls*, Vol. 1 and 2, July 1989. 220 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes*, Vol. 1: *General Information*, April 1991. 172 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes*, Vol. 2: *Walls No. 4 and 6 (Group 1)*, April 1991. 267 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes*, Vol. 3: *Walls No. 8, 9, 10 and 11 (Group 2)*, April 1991. 310 pgs.

3.2(b)-2: Blondet, M., and R. L. Mayes, *The Transverse Response of Clay Masonry Walls Subjected to Strong Motion Earthquakes*, Vol. 4: *Walls No. 3, 5, and 7 (Group 3)*, April 1991. 256 pgs.

4.1-1: He, L., and M. J. N. Priestley, *Seismic Behavior of Flanged Masonry Shear Walls*, May 1988. 119 pgs.

4.1-2: He, L., and M. J. N. Priestley, *Seismic Behavior of Flanged Masonry Shear Walls - Final Report*, November 1992. 279 pgs.

4.2-1: Hegemier, G., and H. Murakami, *On the Behavior of Floor-to-Wall Intersections in Concrete Masonry Construction: Part I: Experimental.*

4.2-2: Hegemier, G., and H. Murakami, *On the Behavior of Floor-to-Wall Intersections in Concrete Masonry Construction: Part II: Theoretical.*

5.1-1: Porter, M., and A. Sabri, *Plank Diaphragm Characteristics*, July 1990. 226 pgs.

5.2-1: Porter, M., F. Yeomans, and A. Johns, *Assembly of Existing Diaphragm Data*, July 1990. 142 pgs.

6.2-1: Scrivener, J., *Bond of Reinforcement in Grouted Hollow-Unit Masonry: A State-of-the-Art*, June 1986. 53 pgs.

6.2-2: Soric, Z., and L. Tulin, *Bond Splices in Reinforced Masonry*, August 1987. 296 pgs.

7.1-1: Paulson, T., and D. Abrams, *Measured Inelastic Response of Reinforced Masonry Building Structures to Earthquake Motions*, October 1990. 294 pgs.

8.1-1: Hart, G., *A Limit State Design Method for Reinforced Masonry*, June 1988.

8.1-2: Hart, G., *Expected Value Design in the Context of a Limit Sate Design Methodology*, February 1990.

8.2-1: Hart, G., and G. T. Zorapapel, *Reliability of Concrete Masonry Wall Structures*, December 1991. 229 pgs.

8.2-2: Hart, G., and N. Sajjad, *Confinement in Concrete Masonry*, December 1990.

8.2-3: Hart, G., and J. Jang, *Seismic Performance of Masonry Wall Frames*, December 1991.

9.1-1: Kariotis, J. C., and A. W. Johnson, *Design of Reinforced Masonry Research Building*, September 1987. 42 pgs.

9.1-2: Kariotis, J. C., and O. M. Waqfi, *Trial Designs Made in Accordance with Tentative Limit States Design Standards for Reinforced Masonry Buildings*, February 1992. 184 pgs.

9.2-1: Seible, F., *Report on Large Structures Testing Facilities in Japan*, September 1985. 120 pgs.

9.2-2: Seible, F., *Design and Construction of the Charles Lee Powell Structural Systems Laboratory*, November 1986. 65 pgs.

9.2-3: Seible, F., *The Japanese Five-Story Full Scale Reinforced Masonry Building Test*, January 1988. 100 pgs.

9.2-4: Seible, F., G. A. Hegemier, M. J. N. Priestley, G. R. Kingsley, A. Kurkchubasche, and A. Igarashi, *The U.S. - TCCMAR Five-Story Full Scale Masonry Research Building Test - Preliminary Report*, October 1992. 58 pgs.

11.1-1: TCCMAR, *Summary Report: U.S. Coordinated Program for Masonry Building Research*, September 1985 to August 1986. 190 pgs.

11.1-2: TCCMAR, *Status Report: U.S. Coordinated Program for Masonry Building Research*, November 1988. 170 pgs.

Appendix to Chapter 8 Commentary

ALTERNATIVE MASONRY STRUCTURE DESIGN REQUIREMENTS

8A.1.2 REFERENCE DOCUMENTS: This section references the *Building Code Requirements for Masonry Structures* (Ref. 8A-1), 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 530.1/ASCE 6/TMS 602). These design and construction documents reference nationally recognized testing standards and material standards developed by the American Society for Testing and Materials (ASTM) and others.

Concern has been expressed about the area of vertical reinforcement permitted in Sec. 3.1.2 of Ref. 8A-1. The percentage of the area of the grout space (minimum grout area) and the cover and clearance requirements in Chapter 8 of Ref. 8A-1 provide reasonable assurance that the strength of the reinforcement can be developed.

8A.1.2.1 Modifications to Appendix A of Reference 8A-1: Appendix A provisions of ACI 530/ASCE 5/TMS 402, "Special Provisions for Seismic Design," are based on seismic zones defined by ASCE 7, *Minimum Design Loads for Buildings and Other Structures*. To be consistent with the *NEHRP Recommended Provisions*, Table 8A.1.1 correlates seismic zones to seismic performance categories.

8A.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. 8A-1 multiplied by a 1.33 factor for load combinations that include wind or earthquake (Ref. 8A-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 strength reduction factor, ϕ, to achieve a reliable design level value.

8A.2.1: Splice length of reinforcement is based on the allowable stress in the reinforcement in accordance with Ref. 8A-1, Sec. 8.5.7. This allowable stress is not modified by the 2.5 factor from Sec. 8A-2 or by a strength reduction factor, ϕ. Splice lengths required by these provisions are therefore identical to the splice length required by Ref. 8A-1.

8A.3 RESPONSE MODIFICATION COEFFICIENTS: Masonry designed in accordance with Chapter 7 of Ref. 8A-1 is required to have 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 2.2.2 for reinforced masonry. Unreinforced masonry shear walls designed in accordance with Chapter 6 of Ref. 8A-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.

8A.4 SEISMIC PERFORMANCE CATEGORY A: Ref. 8A-1 permits three design methods for masonry:

1. Design allowing tensile stresses in masonry (Chapter 6, Reinforced Masonry),

2. Design neglecting tensile strength of masonry (Chapter 7, Unreinforced Masonry), and

3. Empirical design of masonry (Chapter 9, Empirical).

Any of the three methods are considered appropriate for designs in Category A.

8A.5 SEISMIC PERFORMANCE CATEGORY B: Masonry may be designed by Methods 1, 2, or 3 described above; however, in Category B, design of the basic structural system must be based on a structural analysis in accordance with Methods 1 or 2 described above.

8A.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. 8A-1. Further, noncomposite wythes (i.e., cavity walls) and screen walls must meet the detailing requirements of Sec. 8A.6.1.1 and Sec. 8A.6.1.2, respectively.

8A.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. 8A.7.2. Special inspection is required in accordance with Sec. 1.6.2.5.

8A.8 SEISMIC PERFORMANCE CATEGORY E: The additional requirements of Category E are intended to ensure that the structure remains functional after the earthquake.

Chapter 9 Commentary

WOOD STRUCTURE DESIGN REQUIREMENTS

9.1 REFERENCE DOCUMENTS: Unlike concrete and steel, wood construction practices have not been codified in a form that is standard throughout the country. Heavy timber design practice generally follows the *National Design Specification for Wood Construction* (NDS), Ref. 9-1. Conventional light frame construction practice generally follows the provisions of the *One and Two Family Dwelling Code*, Ref. 9-8, jointly sponsored by the three model code groups. This code is a revised and updated version of the Federal Housing Administration's (FHA) *Minimum Property Standards*.

References 9-11 and 9-12 indicate that the term "structural-use panel" has replaced the term "plywood" and this change in terminology was incorporated in the 1991 *Provisions* and is continued in this 1994 edition. The term "structural-use panel" includes wood-based products manufactured to meet a performance standard (Ref. 9-11). One requirement of this performance standard is bracing or lateral force resistance capability. These products include oriented strand board (OSB) and plywood.

Many wood frame structures are a combination of heavy timber and "conventional" light frame construction and also this combination together with other materials (American Institute of Timber Construction, 1985; Breyer, 1993; Faherty and Williamson, 1989; Hoyle and Woeste, 1989; Somayaji, 1992; Stalnaker and Harris, 1989). The provisions of Chapter 25 of the *Uniform Building Code* (UBC) were used as a resource in developing the requirements introduced in the 1991 *Provisions* and further modified in this edition.

The general provisions of Chapter 9 cover construction practices necessary to provide a performance level of seismic resistance consistent with the purposes stated in Chapter 1. These provisions also may be related to gravity load capacity and wind force resistance which is a natural outgrowth of any design procedure.

9.2 STRENGTH OF MEMBERS AND CONNECTIONS: The seismic force provisions of Chapter 2 were developed on the premise of the resistance capacity of members and connections at the yield level (ASCE Committee on Wood Load and Resistance Factor Design for Engineered Wood Construction, 1988; Canadian Wood Council, 1990; Keenan, 1986) whereas typical design for wood frame structures has used "allowable" stresses and implied factors of safety. To accommodate this difference in philosophy, adjustments were made to tabulated "allowable" stresses in the reference documents. The adjustment has been set at 216 percent of basic "allowable" stresses or capacities 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. The 216 percent figure comes from the extensive in-grade testing program recently conducted in connection with the development of the wood load and resistance factor design (LRFD) document. The capacity reduction factors have been revised to be compatible with the values in that document with adjustments made to provide approximate equivalency with existing allowable capacities in use in the model codes. The values for connections were adjusted and a new factor for bolts in single shear members subject to seismic cyclic forces was added to recognize the results of testing and the poor performance of such connections in the January 1994 Northridge earthquake.

Wood has had a variety of "load" factors and many of the accepted allowable stresses do not have a consistent relationship to an elastic limit or even an ultimate limit. When making the judgment for setting the adjustment factor and capacity reduction factors 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 and comparison with capacities required using existing model codes..

9.3 SEISMIC PERFORMANCE CATEGORY A: Wood frame buildings assigned to Category A other than one- and two-family dwellings must conform with Sec. 9.10 or comply with Sec. 2.2.5.1. One- and two-family detached dwellings are exempt from any requirements.

9.4 SEISMIC PERFORMANCE CATEGORY B: Buildings assigned to Category B are required to meet the minimum construction requirements of Sec. 9.10 (Sherwood and Stroh 1989) or must be engineered using standard design methods and principles of mechanics. Conventional light timber frame construction requirements were modified in the 1991 *Provisions* to limit the spacing between braced wall lines based on calculated capacities to resist the loads and forces imposed. Wood moment resisting frames are designed to resist both vertical loads and lateral forces. More recent construction techniques that utilize wood for lateral force resistance have been in the form of diaphragms or shear walls which are discussed further in Sec. 9.9. Limitations have been set on the use of wood diaphragms that are used in combination with concrete and masonry walls or where torsion is induced by the arrangement of the vertical resisting elements. Connectors and other proprietary hardware for wood framing have been tested in laboratories and actual structures many times over the years and many provide adequate performance when properly installed. There are many references (Department of the Army, Navy and Air Force, 1980; Forest Products Laboratory, 1986; Goetz et al., 1989) that describe the engineering practices and procedures used to design wood buildings that will perform adequately when subjected to lateral forces. The list at the end of this *Commentary* chapter give some, but by no means all, of these.

9.5 SEISMIC PERFORMANCE CATEGORY C: Buildings assigned to Category C are required to meet additional requirements beyond those prescribed for Categories A and B. There are some limitations placed on materials and the use of diaphragm nailing to transfer concrete and masonry wall anchorage forces is prohibited.

Conventional light timber framing construction has additional limitations placed on height and braced wall spacing.

9.6 SEISMIC PERFORMANCE CATEGORY D: The additional requirements for buildings assigned to Category D correspond roughly to the requirements for ordinary wood frame construction in high seismicity areas of the United States. There is a further limitation placed on the height of conventional light timber frame structures. Wall sheathing materials other than blocked structural-use panels or wood sheathing are permitted only for conventional construction and are limited to one-story buildings or the top story of taller buildings. Buildings in Seismic Hazard Exposure Groups II and III are required to be designed in accordance with the Sec. 9.8.

The lack of adequate cyclic or dynamic test data on sheathing materials other than wood or structural-use panels and the observed poor performance in past earthquakes of structures utilizing other materials for lateral resistance dictates that there be limits on the application of these other materials. Recent cyclic testing of gypsum wallboard sheathed panels has confirmed that the strength of the assembly degrades.

9.7 SEISMIC PERFORMANCE CATEGORY E: The requirements for buildings assigned to Category E limit shear wall sheathing to blocked structural-use panels or wood sheathing and prescribe blocked diaphragms. Structural-use panels must be applied directly to the framing members; the use of gypsum wallboard between the structural-use panels and the framing members is prohibited because of the poor performance of nails in gypsum. Restrictions on allowable shear values for structural-use shear panels when used in conjunction with concrete and masonry walls are intended to provide for deformation compatibility of the different materials. All buildings in this category are required to conform to Sec. 9.8 since this category applies to essential facilities in the areas of highest seismic exposure in the United States.

The greater need for reliable performance of structures in this category requires that diaphragm and shear wall materials be restricted to those for which there is adequate knowledge of behavior under dynamic forces.

9.8 ENGINEERED TIMBER CONSTRUCTION: Engineered construction for wood buildings as defined by these provisions encompasses all buildings that cannot be classified as conventional construction. Therefore, any building exceeding the heights or having braced walls spaced at greater intervals than prescribed in Table 9.10.1-1a or not conforming to the requirements in Sec. 9.10 must be engineered using standard design methods and principles of mechanics. Framing members in engineered wood construction are sized based on calculated capacities to resist the loads and forces imposed. More recent construction techniques that utilize wood for lateral force resistance have been in the form of diaphragms or shear walls are discussed further in Sec. 9.9. Limitations have been set on the use of wood diaphragms that are used in combination with concrete and masonry walls or where torsion is induced by the arrangement of the vertical resisting elements. Connectors and other proprietary hardware for wood framing have been tested in laboratories and actual structures many times over the years and many provide adequate performance when properly installed. Wood moment resisting frames are designed to resist both vertical loads and lateral forces. Detailing at columns to beam/girder connections is critical in developing frame action and must incorporate effects of member shrinkage.

There are many references that describe the engineering practices and procedures used to design wood buildings that will perform adequately when subjected to lateral forces. The list at the end of this *Commentary* chapter gives some, but by no means all, of these.

9.8.1 FRAMING REQUIREMENTS: All framing that is designed as part of an engineered wood building must be designed with connectors that are able to transfer the required forces between various components. These connectors can be either proprietary hardware or some of the more conventional connections used in wood construction. However, the capacity of these connectors should be designed according to accepted engineering practice to ensure that they will have the capacity to resist the forces.

9.8.2 DIAPHRAGM AND SHEAR WALL REQUIREMENTS: All diaphragms and shear walls are to be designed according to accepted engineering practice which includes procedures by the American Plywood Association, the *National Design Specification*, or the various building codes. See *Commentary* Sec. 9.9.

9.8.2.1 Framing: Minimum member thickness of 2 in. (51 mm) is specified to provide adequate edge distance and prevent splitting of the member. Where close nail spacing is used, where longer than normal penetration will occur or where multiple lines of fasteners are used, the member thickness must be increased.

The edge or boundary members of diaphragms and shear walls act as chords and collectors transmitting lateral forces. These elements must be designed for the axial forces induced in them. The splices that normally will occur in many of these members need to be carefully detailed. The forces that are induced in members at the edges of openings in walls and diaphragms need to be transferred into the body of the diaphragm or shear wall beyond the edges of the openings.

9.8.2.2 Anchorage and Connections: The anchorage connections used in engineered wood construction must be capable of resisting the forces that will occur between adjacent members such as diaphragms and shear walls. These connections can utilize proprietary hardware or be designed in accordance with principles of mechanics. Connections are often the cause of structural failures in wood buildings, and the engineer is cautioned to use conservative values for allowable capacities since most published values are based on monotonic, not cyclic, load applications (U.S. Department of Agriculture, National Oceanic and Atmospheric Administration, 1971). Testing has shown that one-sided bolted connections subject to cyclic loading, such as hold-down devices, do not perform. This was substantiated by the poor performance of various wood frame elements in structures in the January 1994 Northridge earthquake. A logical load path for the structure must be provided so that the forces experienced by the upper portions of the structure are transmitted adequately through the lower portions of the structure to the foundation.

Concrete or masonry wall anchorages using toe nails or nails subject to withdrawal are prohibited by these provisions. It has been shown that these types of connection are inadequate and do not perform well (U.S. Department of Agriculture, National Oceanic and Atmospheric Administration, 1971). Ledgers subjected to cross-grain bending or tension perpendicular to grain also have fared poorly in past earthquakes. This use is now prohibited by these provisions.

9.8.2.3 Torsion: Torsional response of the building, due to irregular stiffness at any level within the structure, is a potential cause of failure. Horizontal diaphragms must be rigid relative to the vertical resisting elements to be capable of distributing torsion; flexible diaphragms have very limited capacity for distributing torsional forces and, therefore, the aspect ratios are limited by the provisions. Flexible diaphragms are not permitted in buildings over two stories in height with concrete or masonry walls. See American Plywood Association (1991) and Applied Technology Council (1981) for information on determination of approximate deformation of horizontal plywood diaphragms. Rational methods such as standard engineering mechanics methods may be used to determine the deflections for diaphragms at any given level in the structure. In situations where a series of vertical lateral force resisting elements (shear walls) are aligned in a single row, and the panels are constructed of materials with differing stiffnesses such as concrete or masonry and wood frame, the more rigid panels will attract the major portion of the seismic forces. The forces will be distributed to the different elements according to their relative stiffnesses. If necessary, the stiffness of the wood frame panels can be increased with the use of adhesives. Where adhesives are used, see *Commentary* Sec. 9.9. However, it is noted that there are no rational methods for determining deflections in diaphragms that are constructed with non-wood sheathing materials. The torsional behavior of a structure is sometimes due to differences in stiffness between two parallel lines of resistance, and therefore, when the ratio of the two stiffnesses is greater than 4 to 1, additional requirements for bracing walls and diaphragms are included. If the nail stiffness values or shear stiffness of non-wood sheathing materials is determined in a scientific manner, such as through experimental testing, the calculations for determining the stiffness of shear panels will be considered validated.

9.9 DIAPHRAGMS AND SHEAR WALLS: Many wood-framed buildings resist earthquake--generated forces by acting as a "box system." The forces are transmitted through horizontal diaphragms, such as roofs and floors, to reactions provided by shear walls. The forces are, in turn, transmitted to the lower stories and to the final point of resistance, the foundations. A shear wall is a vertical diaphragm generally considered to act as a cantilever from the foundation.

A diaphragm is a nearly horizontal structural unit which acts as a deep beam or girder. The analogy to a girder is somewhat more appropriate since girders and diaphragms are made up as assemblies (American Plywood Association, 1991; Applied Technology Council, 1981). Sheathing acts as the "web" to resist the shear in diaphragms and is stiffened by the framing members, which also provide support for gravity loads. Flexure is resisted by the edge elements acting like "flanges" to resist induced tension or compression forces. The "flanges" may be top plates, ledgers, bond beams or any other continuous element at the perimeter of the diaphragm. A load path must be provided to transmit the lateral forces from the diaphragm through the vertical resisting elements to the foundation. It is important for the designer to follow the forces down, as he would for gravity loads, designing each connection and member as he proceeds along the load path.

The "flange" (chord) can serve several functions at the same time providing resistance to loads and forces from different sources. When it functions as the tension or compression flange of the "girder," it is important that the connection to the "web" be designed to accomplish the shear transfer. Since most diaphragm "flanges" consist of many pieces, it is important that the splices be designed to transmit the tension or compression occurring at the location of the splice and to recognize that the direction of application of earthquake forces can reverse. It should also be recognized that the shear walls parallel to the flanges may be acting with the flanges to distribute the diaphragm shears. When earthquake forces are delivered at right angles to the direction considered previously, the "flange" becomes a part of the reaction system. It may function to transfer the diaphragm shear to the shear wall(s), either directly or as a "drag strut" between segments of shear walls that are not continuous along the length of the diaphragm.

For shear walls, which may be considered as deep vertical cantilever beams, the "flanges" are subjected to tension and compression while the "webs" resist the shear. It is important that the "flange" members, splices at intermediate floors, and the connection to the foundation be detailed and sized for the induced forces. The shear wall aspect ratios, h/w, have been limited to 2/1 in light of the poor performance of walls with larger aspect ratios in recent tests and in the January 1994 Northridge earthquake.

The "webs" of diaphragms and shear walls often have openings. The transfer of forces around openings can be treated similarly to openings in the webs of steel girders. Members at the edges of openings have forces due to flexure and the higher web shear induced in them and the resultant forces must be transferred into the body of the diaphragm beyond the opening.

In the past, wood sheathed diaphragms have been considered to be flexible by many designers and code enforcement agencies. The newer versions of the codes now recognize that the determination of rigidity or flexibility for determination of how forces will be distributed is dependent on the relative deformations of the horizontal and vertical resisting elements. Wood sheathed diaphragms in buildings with wood frame shear walls with various types of sheathing may be relatively rigid compared with the vertical resisting system and, therefore, capable of transmitting torsional lateral forces. A relative deformation of the diaphragm of two or more compared with the vertical resisting system deformation under the same force is used to define a diaphragm as being flexible.

Discussions of these and other topics related to diaphragm and shear wall design, such as pitched or notched diaphragms, may be found in the references.

The capacity of shear walls will be determined either from tabulated values that are based on experimental results or from standard principles of mechanics. The tables of allowable values for shear walls sheathed with other than wood or wood-based structural use panels have been eliminated in the 1991 *Provisions* as a result of re-learning the lessons from past earthquakes and testing on the performance of buildings sheathed with these materials during the Northridge earthquake. One stipulation is that there are no accepted rational methods for calculating deflections for shear walls and diaphragms that are sheathed with materials other than wood-based structural-use panel products fastened with nails. Therefore, if a rational method is to be used, the capacity of the fastener in the sheathing material must be validated by acceptable test procedures employing cyclic forces or displacements. Validation must include correlation between the overall stiffness and capacity predicted by principles of mechanics and that observed from test results. The standard ASTM E564 test procedure or its equivalent, using cyclic loading, would be considered acceptable. A shear wall or diaphragm sheathed with dissimilar materials on the two faces should be designed as a single-sided wall using the capacity of the stronger of the materials and ignoring the weaker of the materials.

The provisions are based on assemblies having energy dissipation capacities which were recognized in setting the R factors. For diaphragms and shear walls utilizing wood framing the energy dissipation is almost entirely due to nail bending. Fasteners other than nails and staples have not been extensively tested under cyclic load application. When screws or adhesives have been tested in assemblies subjected to cyclic loading, they have had a brittle mode of failure. For this reason, adhesives are prohibited for wood framed shear wall assemblies and only the tabulated values for nailed or stapled sheathing is recommended. If one wished to use shear wall sheathing attached with adhesives, caution should be used (Dolan and White, 1992; Foschi and Filiatrault, 1990). The increased stiffness will result in larger forces being attracted to the structure. The anchorage connections and adjoining assemblies must, therefore, be designed for these increased forces. Due to the brittle failure mode, these walls should be designed to remain elastic, similar to unreinforced masonry. The use of adhesives for attaching sheathing for horizontal diaphragms improves their performance.

9.10 CONVENTIONAL LIGHT TIMBER CONSTRUCTION: These provisions intend that a building using conventional construction methods and complying with the requirements of this section be deemed capable of resisting the seismic lateral forces imposed by the *Provisions*.

Repetitive framing members such as joists, rafters and studs together with sheathing and finishes comprise conventional light timber construction. The subject of conventional construction is addressed in each of the model codes. It is acknowledged and accepted that, for the most part, the conventional construction provisions in the model codes concerning framing members and sheathing that carry gravity loads are adequate. This is generally because the tables in the codes giving allowable spans have been developed using basic principles of mechanics. For seismic lateral force resistance, however, experience has shown that additional requirements are needed.

To provide lateral force resistance in vertical elements of buildings, wall bracing requirements have been incorporated in conventional construction provisions of the model codes. These have generally, with a few exceptions, been adequate for single family residences for which conventional construction provisions were originally developed. While the model codes have been quite specific as to the type of bracing materials to be used and the amount of bracing required in any wall, no limits on the number or maximum separation between braced walls have been established. This section of the provisions introduces a new concept of mandating the maximum spacing of braced wall lines. By mandating the maximum spacing of braced wall lines and thereby limiting the lateral forces acting on these vertical elements, these revisions provide for a lateral force resisting system that will be less prone to over-stressing and can be applied and enforced more uniformly than previous code provisions. While specific elements of light frame construction may be calculated to be over-stressed, there is typically a great deal of redundancy and uncounted resistance in such structures and they have generally performed well in past earthquakes. The experience in the Northridge earthquake was, however, less reassuring, especially for those residences relying on gypsum board or stucco for lateral force resistance. The light weight of conventional construction together with the large energy dissipation capacity of the multiple fasteners used are major factors in the observed good performance where wood or wood-based panels were used.

9.10.1.1 Braced Wall Spacing: Table 9.10.1-1a has been added to this section and prescribes the spacing of braced walls and number of stories permitted for conventional construction structures. Figures C9.10.1.1-1 and C9.10.1.1-2 illustrate the basic components of the lateral bracing system. Information in Tables 9.10.1-1a and 1b was in the 1991 Edition.

9.10.1.2 Braced Wall Sheathing Requirements: Table 9.10.1-1b has been added prescribing the length of bracing along each 25 ft (7.6 m) length of braced wall line. See *Commentary* Sec. 9.9 regarding adhesive attachment. Total height of structures has been reduced to limit overturning on the braced walls so that uplift is not generally encountered. The height limit will accommodate 8 to 10 ft story heights.

9.10.2 WALL FRAMING AND CONNECTIONS: The intent of this section is to rely on the traditional light frame conventional construction materials and fastenings as prescribed in the references for this Chapter. Braced wall panels are not required to be aligned vertically or horizontally but stacking is desirable where possible. With the freedom provided for non alignment it becomes important that a load path be provided to transfer lateral forces from upper levels through intermediate vertical and horizontal resisting elements to the foundation. Connections between horizontal and vertical resisting elements are prescribed. In buildings two stories in height it is desirable to have interior braced panel walls supported on a continuous foundation and for buildings over two stories in height it should be mandatory. See Figures C9.10.2-1 through C9.10.2-11 for examples of connections.

REFERENCES

American Institute of Timber Construction. 1985. *Timber Construction Manual.* New York, New York: John Wiley and Sons, Inc.

American Plywood Association. 1991. *Design/Construction Guide--Diaphragms.* Tacoma, Washington: APA.

ASCE Committee on Wood Load and Resistance Factor Design for Engineered Wood Construction. 1988. *A Pre Standard Report American Society of Civil Engineers.*

Applied Technology Council. 1981. *Guidelines for the Design of Horizontal Wood Diaphragms,* ATC-7. Redwood City, California: ATC.

Breyer, Donald E. 1993. *Design of Wood Structures,* Third Edition. New York, New York: McGraw-Hill Book Company.

Canadian Wood Council. 1990. *Wood Design Manual.* Ottawa: Canadian Wood Council.

Canadian Wood Council. 1991. *Wood Reference Handbook.* Ottawa: Canadian Wood Council.

Department of the Army, Navy and Air Force. 1980. *Seismic Design for Buildings,* TM5-809-10 (Tri-Services Manual). Washington, D.C.: U.S. Government Printing Office.

Dolan, J. D., and M. W. White. 1992. "Design Considerations for Using Adhesives in Shear Walls." *ASCE Journal of Structural Engineering* 118(12):3473-3480.

Faherty, Keith F., and T. G. Williamson. 1989. *Wood Engineering and Construction Handbook.* New York, New York: McGraw-Hill.

Forest Products Laboratory. 1986. *Wood: Engineering Design Concepts.* University Park: Materials Education Council, The Pennsylvania State University.

Foschi, R. O., and A. Filiatrault. 1990. *Performance Evaluation of 3M Scotch Grip Wood Adhesive 5230 for the Static and Dynamic Design of Timber Shear Walls and Diaphragms,* Vancouver: Department of Civil Engineering Report, University of British Columbia.

Goetz, Karl-Heinz, Dieter Hoor, Karl Moehler, and Julius Natterer. 1989. *Timber Design and Construction Source Book: A Comprehensive Guide to Methods and Practice.* New York, New York: McGraw-Hill.

Hoyle and Woeste. 1989. *Wood Technology and Design of Structures.* Iowa State University Press.

Keenan, F. J. 1986. *Limit States Design of Wood Structures.* Morrison Hershfield Limited.

Sherwood and Stroh. 1989. "Wood-Frame House Construction" in *Agricultural Handbook 73*. Washington, D.C.: U.S. Government Printing Office.

Somayaji, Shan. 1992. *Structural Wood Design.* St. Paul, Minnesota: West Publishing Co.

Stalnaker, Judith J., and E. C. Harris. 1989. *Structural Design in Wood.* New York, New York: McGraw-Hill.

U.S. Department of Agriculture, National Oceanic and Atmospheric Administration. 1971. *San Fernando, California, Earthquake of February 9, 1971.*

Additional references are listed in "Bibliography on Lumber and Wood Panel Diaphragms" by John Peterson, published in the American Society of Civil Engineers' *Journal of Structural Engineering*, Vol. 109 No. 12, Dec. 1983.

SEISMIC PERFORMANCE CATEGORY	MAXIMUM WALL SPACING (FEET) *
C,D AND E	25
B	35
A	NOT REQUIRED

*REFER TO TABLE 9.10.1–1B FOR MINIMUM LENGTH OF WALL BRACING

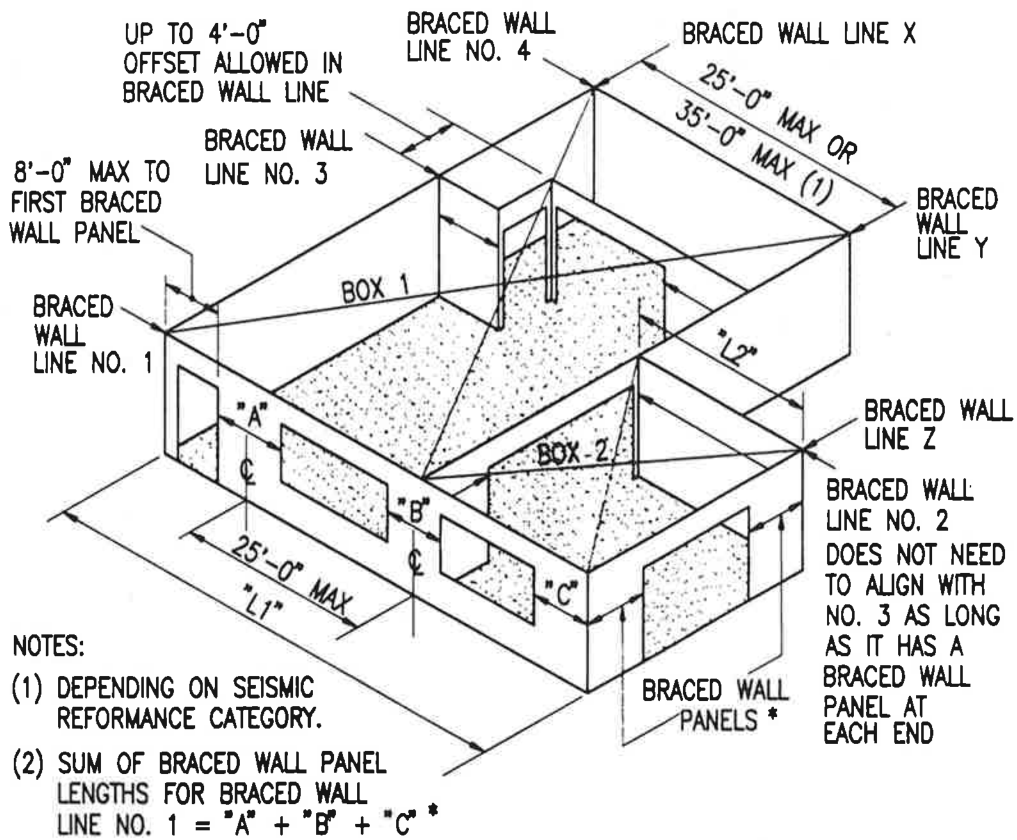


FIGURE C9.10.1.1-1 Acceptable one-story bracing example.

SEISMIC PERFORMANCE CATEGORY	MAXIMUM WALL SPACING (FEET) *
C,D AND E	25
B	35
A	NOT REQUIRED

*REFER TO TABLE 9.10.1-1B FOR MINIMUM LENGTH OF WALL BRACING

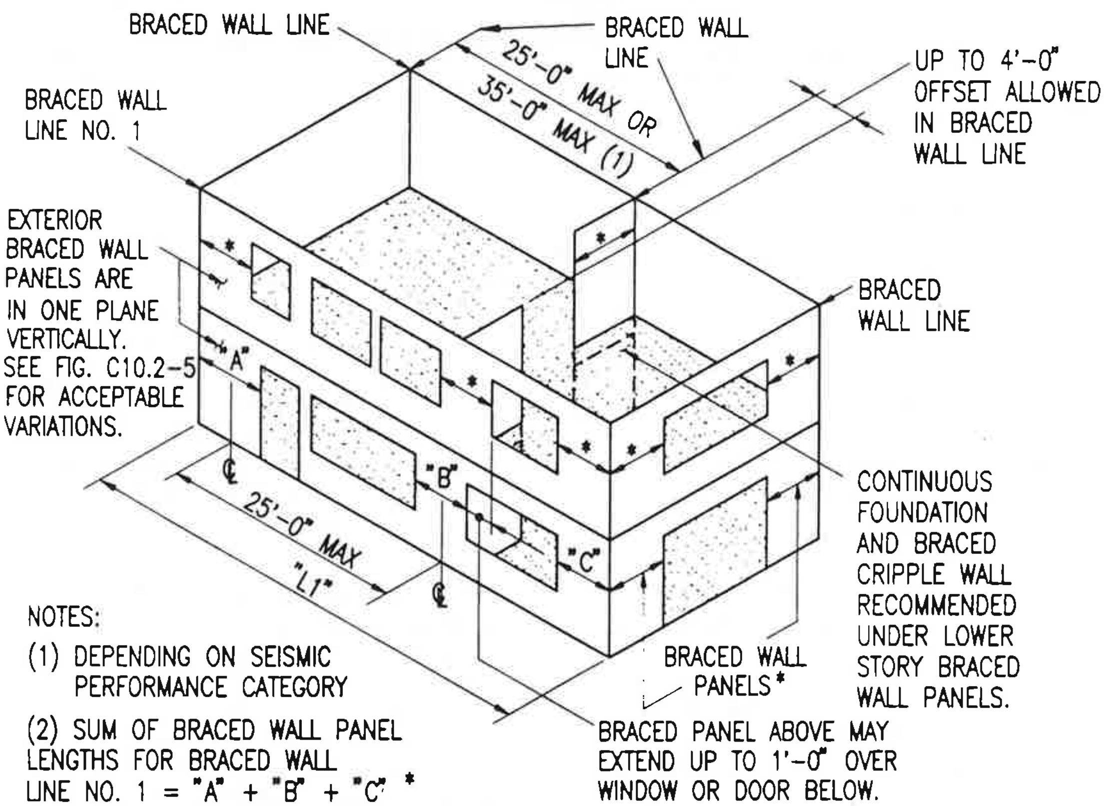
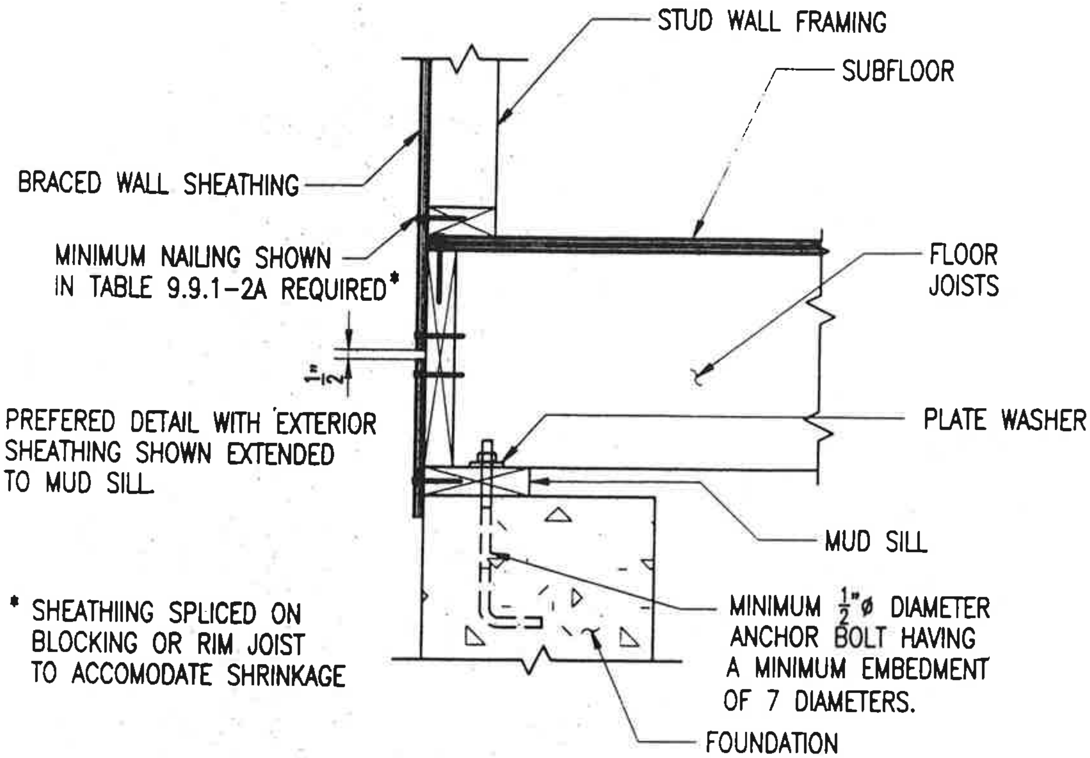


FIGURE C9.10.1.1-2 Acceptable two-story bracing example.



NUMBER OF STORIES	SPACING OF ANCHOR BOLTS (FEET)
ONE OR TWO	6
OVER TWO	4

FIGURE C9.10.2-1 Wall anchor detail.

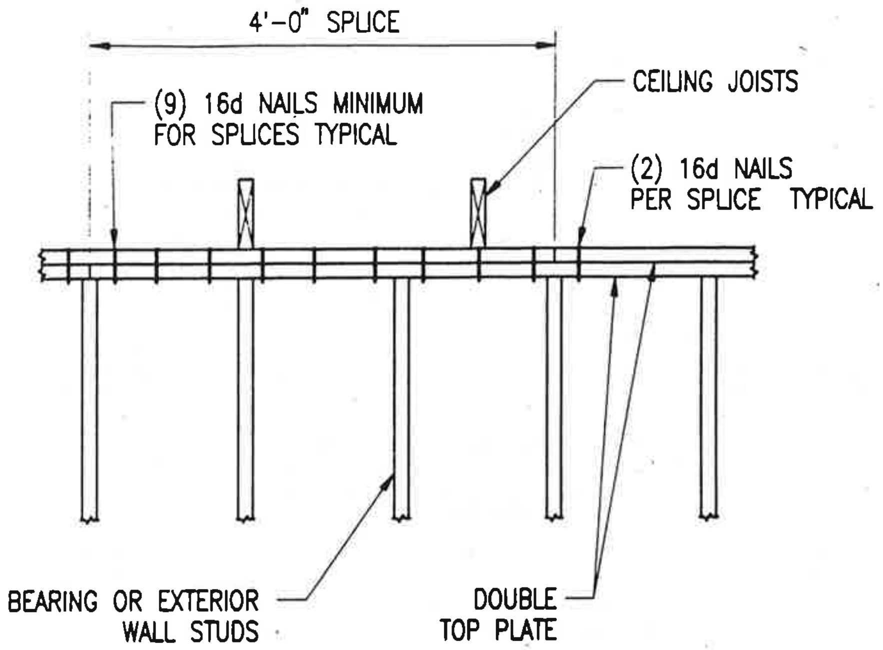


FIGURE C9.10.2-2 Double top plate splice.

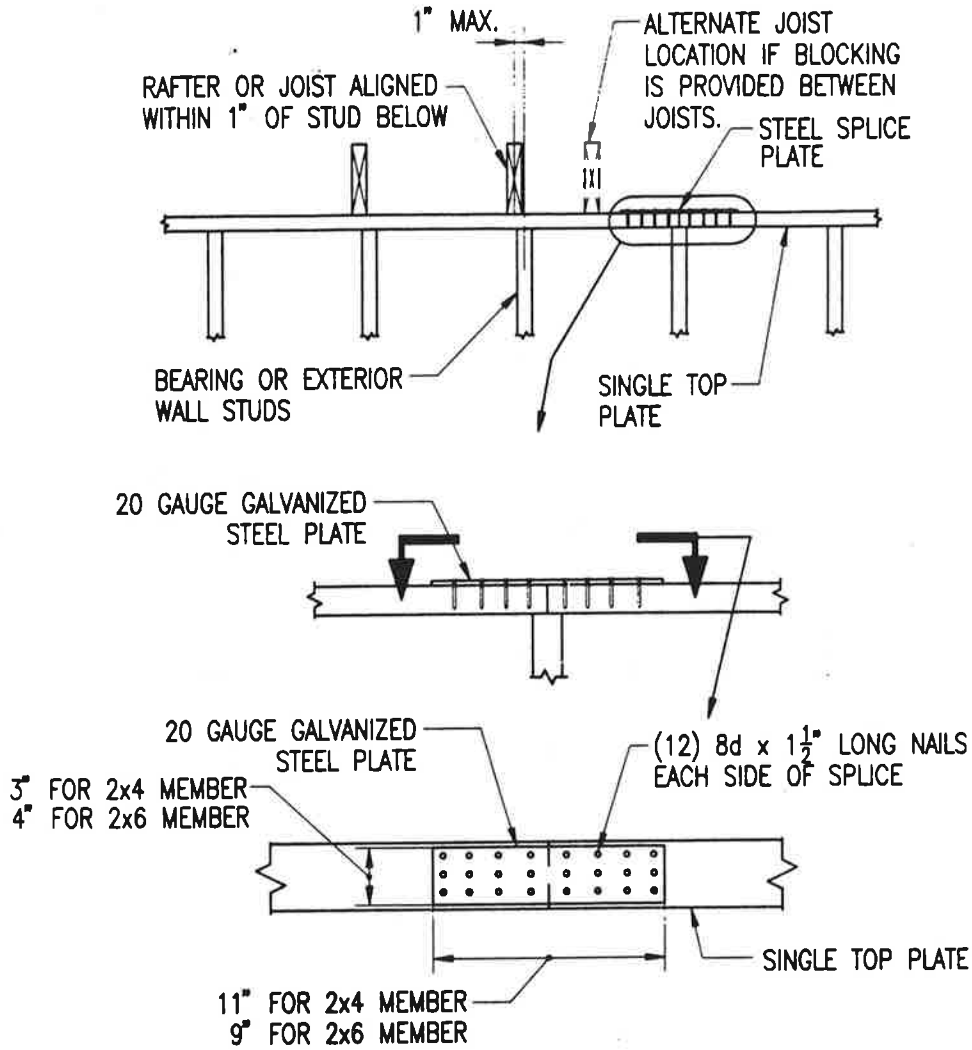
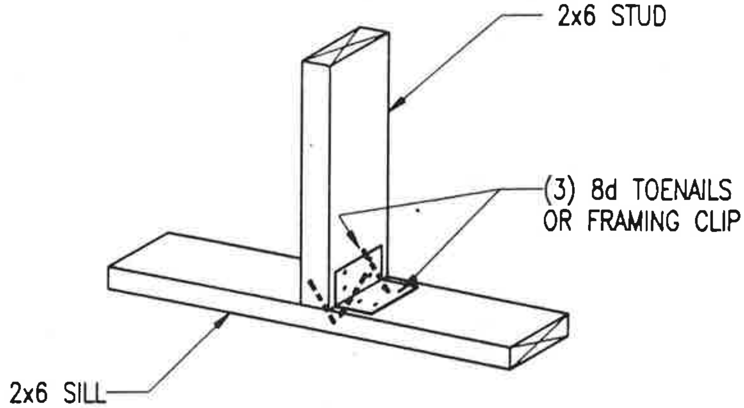
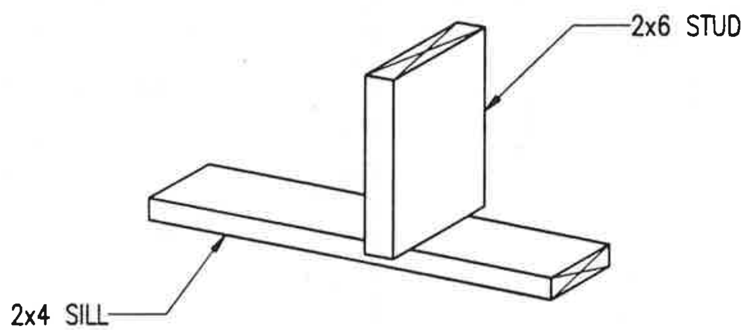


FIGURE C9.10.2-3 Single top plate splice.



ACCEPTABLE FULL BEARING ON BOTTOM PLATE



UNACCEPTABLE BEARING ON BOTTOM PLATE

FIGURE C9.10.2-4 Full bearing on bottom plate.

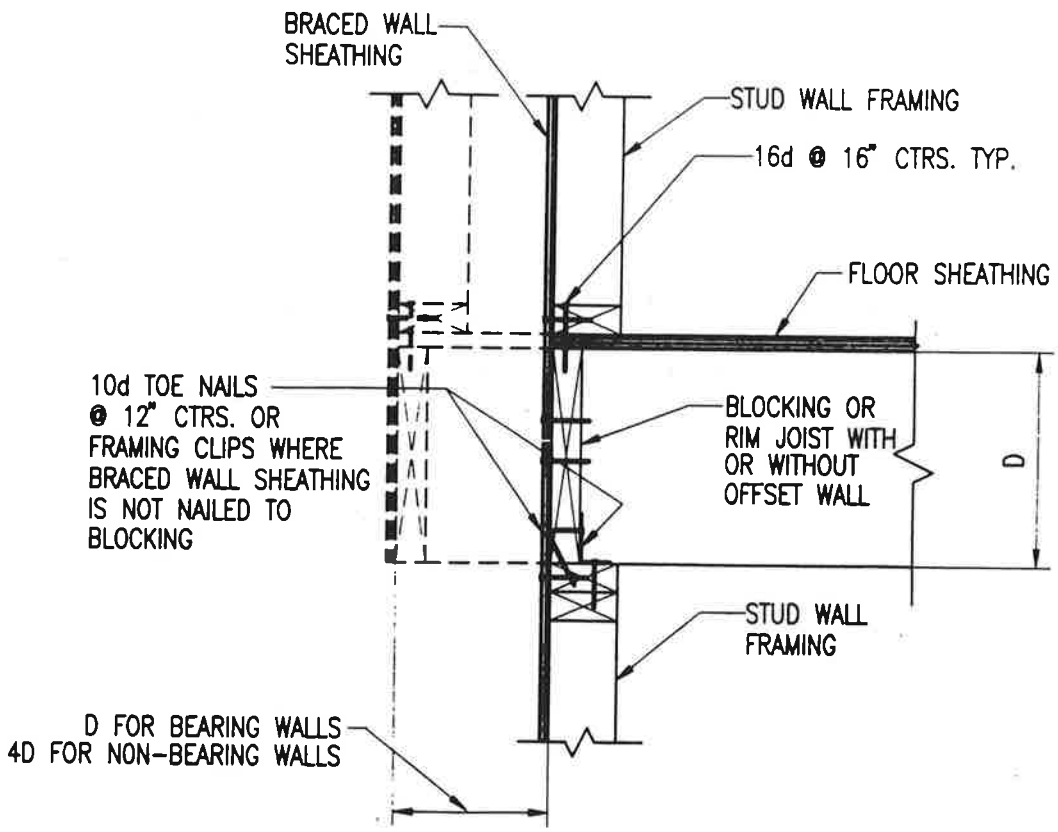


FIGURE C9.10.2-5 Exterior braced wall.

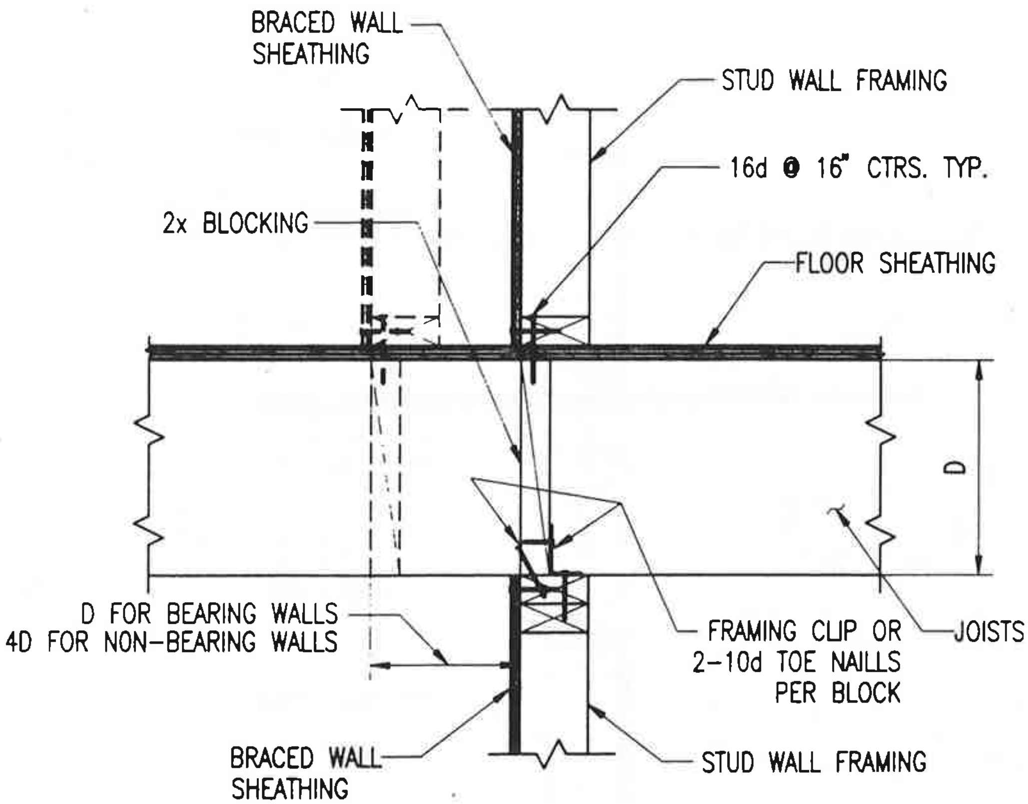


FIGURE C9.10.2-6 Interior braced wall at perpendicular joist.

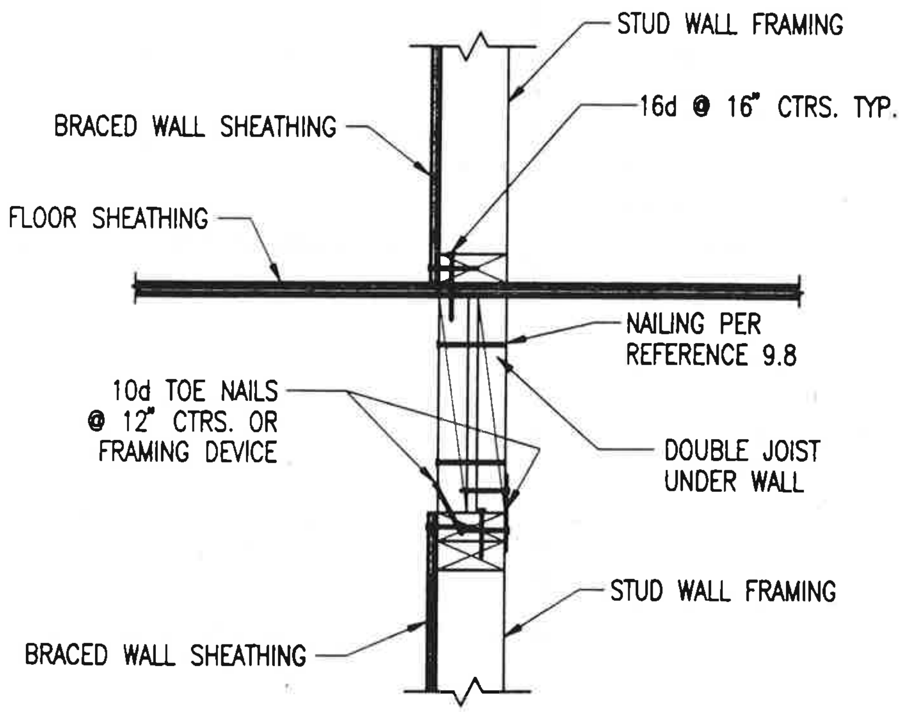


FIGURE C9.10.2-7 Interior braced wall at parallel joist.

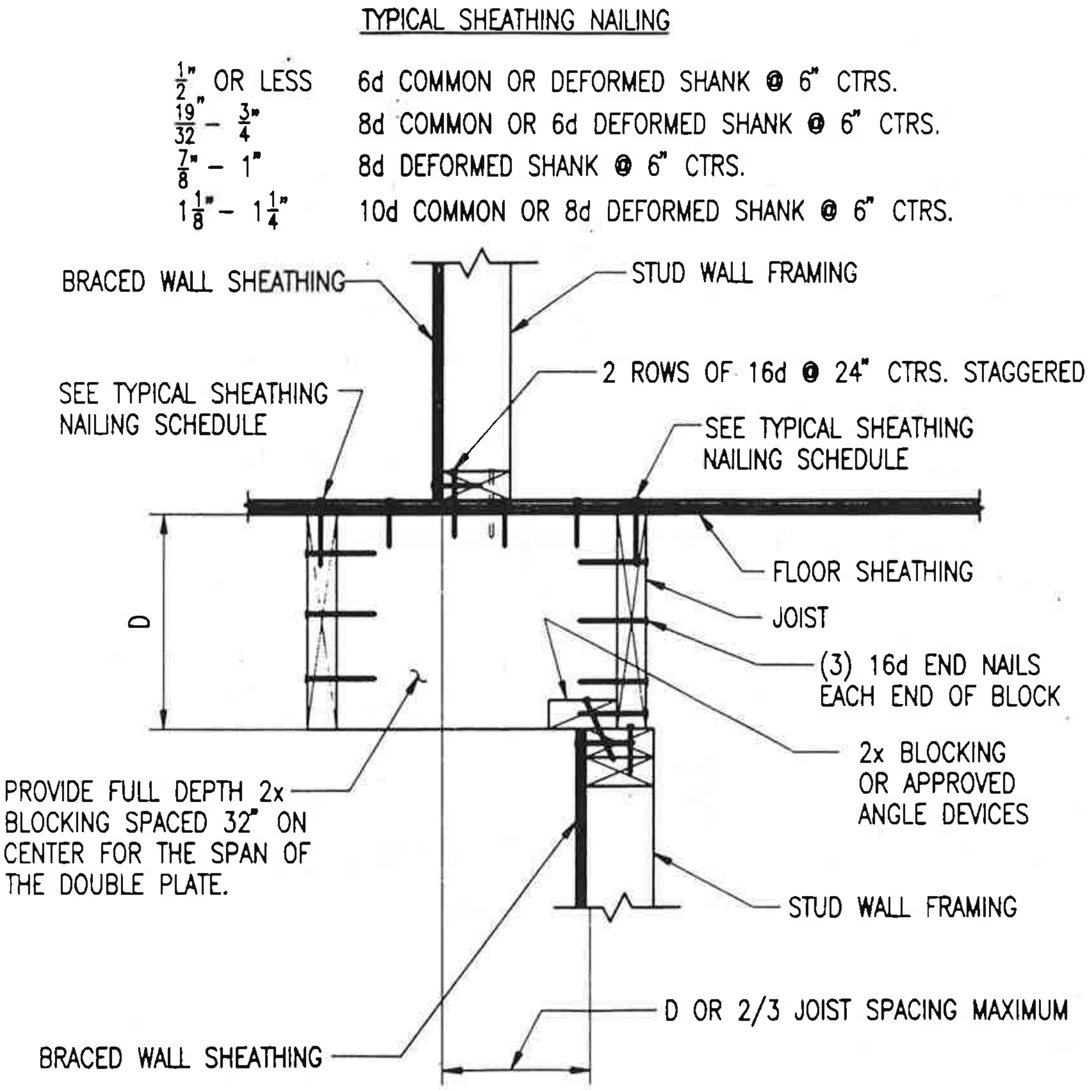


FIGURE C9.10.2-8 Offset at interior braced wall.

TYPICAL SHEATHING NAILING

$\frac{1}{2}$" OR LESS	6d COMMON OR DEFORMED SHANK @ 6" CTRS.
$\frac{19}{32}$" – $\frac{3}{4}$"	8d COMMON OR 6d DEFORMED SHANK @ 6" CTRS.
$\frac{7}{8}$" – 1"	8d DEFORMED SHANK @ 6" CTRS.
$1\frac{1}{8}$" – $1\frac{1}{4}$"	10d COMMON OR 8d DEFORMED SHANK @ 6" CTRS.

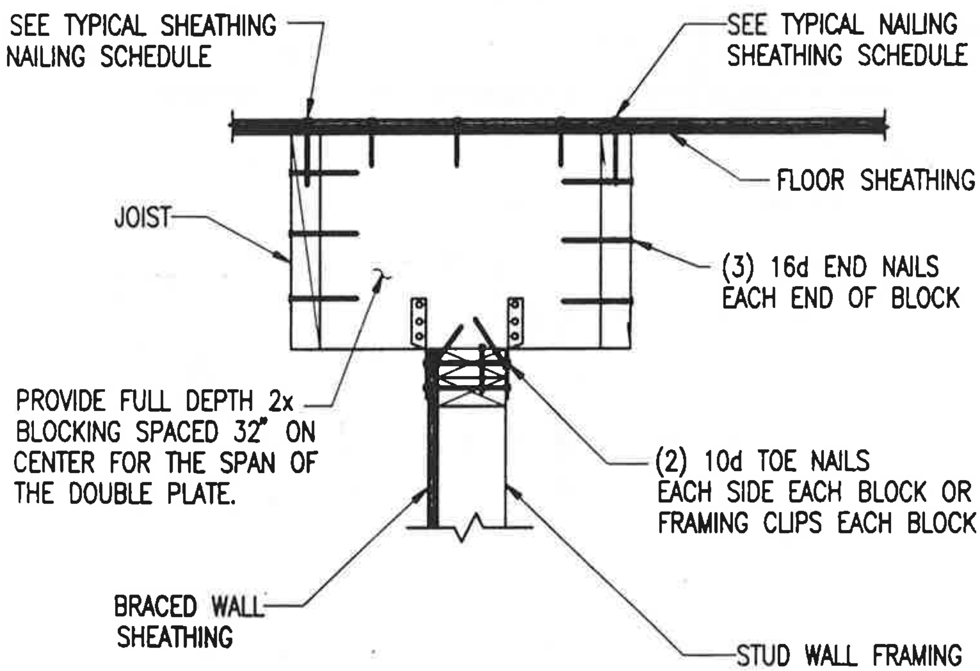


FIGURE C9.10.2-9 Diaphragm connection to braced wall below.

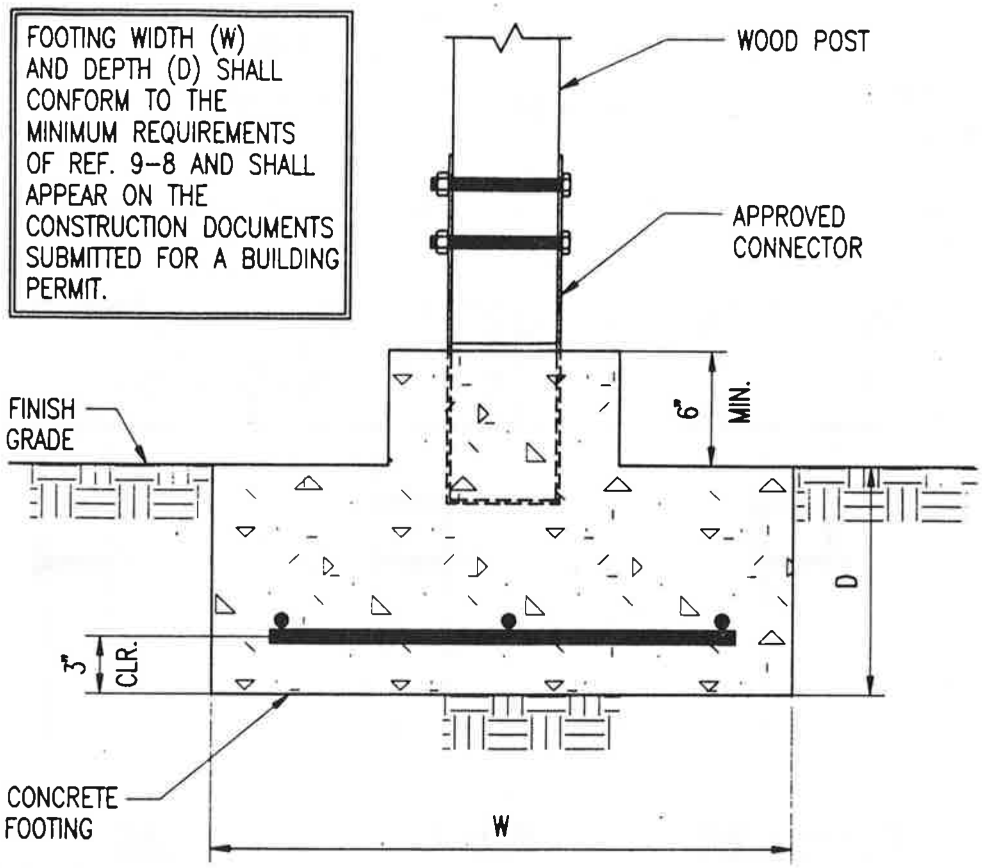


FIGURE C9.10.2-10 Post base detail.

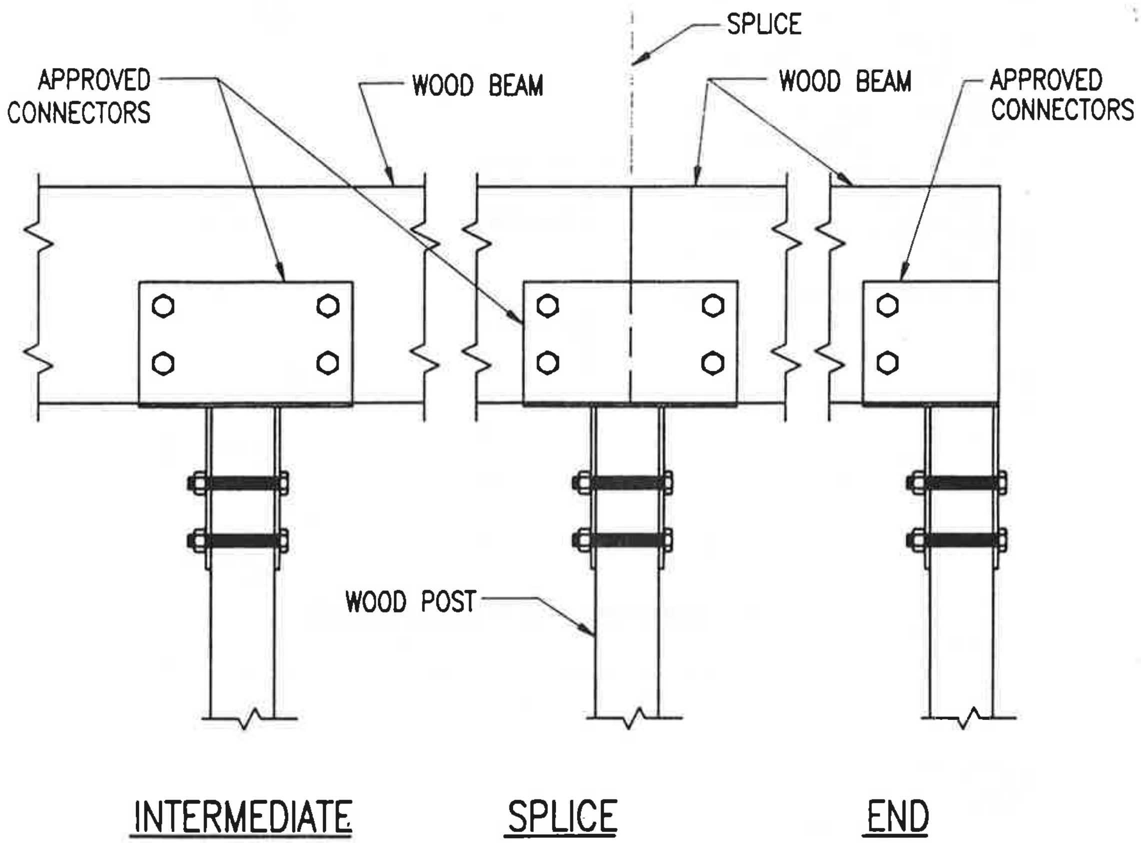


FIGURE C9.10.2-11 Wood beam connection to post.

Appendix A

BACKGROUND FOR MAPS 1 THROUGH 4

This appendix presents presents the background for *Provisions* Maps 1 through 4. It should be noted, however, that the recommended regionalization maps and seismic design coefficients reflect the collective judgment based upon the best scientific knowledge available in 1976, of the committees involved in the Applied Technology Council (ATC) study that led to the development of the preliminary version of the *Provisions*. It was expected 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 were included in the "Appendix to Chapter 1" in the 1988 Edition of the *Provisions*. In the 1991 Edition, that appendix was revised to introduce new spectral maps and procedures for review and comment. For the 1994 Edition, that appendix has again been revised to describe recent and future mapping and design values efforts but improved spectral maps are retained.

This *Commentary* appendix explains the bases for the original recommendations concerning Maps 1 through 4 as a guide both to the user of the *Provisions* and to those who will continue to improve the *Provisions* in the future.

Introduction: 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.

Policy Decisions: The recommended ground-shaking regionalization maps reflect three 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 decision 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 earlier zoning maps based on estimates of the maximum ground-shaking experienced during the recorded historical period without consideration of how frequently such motions might occur. Agreement in the profession with this policy decision is not unanimous. 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 since 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. First, 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. Second, a building properly designed for a particular ground motion will provide considerable protection to the lives of occupants during a more severe ground motion. Third, 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 above discussion refers 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.

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) since 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 the seismic design coefficient. For structures that can safely strain past their yield point, the forces determined in accordance with Sec. 2.3.2 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 when the *Provisions* were developed, 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. Although these parameters do not have precise definitions in physical terms their significance is explained below.

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 damping 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 A-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.

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-

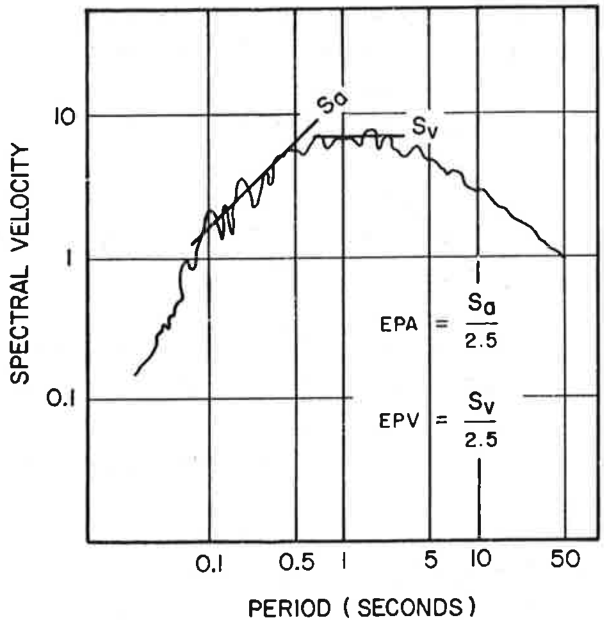


FIGURE A-1 Schematic representation showing how EPA and EPV are obtained from a response spectrum.

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, and these factors tend to produce proportionally larger increases in that portion of the response spectrum represented by the EPV.

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. At present, however, there are 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 site coefficients C_a and C_v in Sec. 1.4.2.3 and the seismic response coefficient in Sec. 2.3.2, 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 in the discussion of "Implied Risk" in *Commentary* Appendix C.

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 A-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:

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

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

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

4. With the foregoing information as input, a computer program based on probabilistic principles can generate values that then are 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.

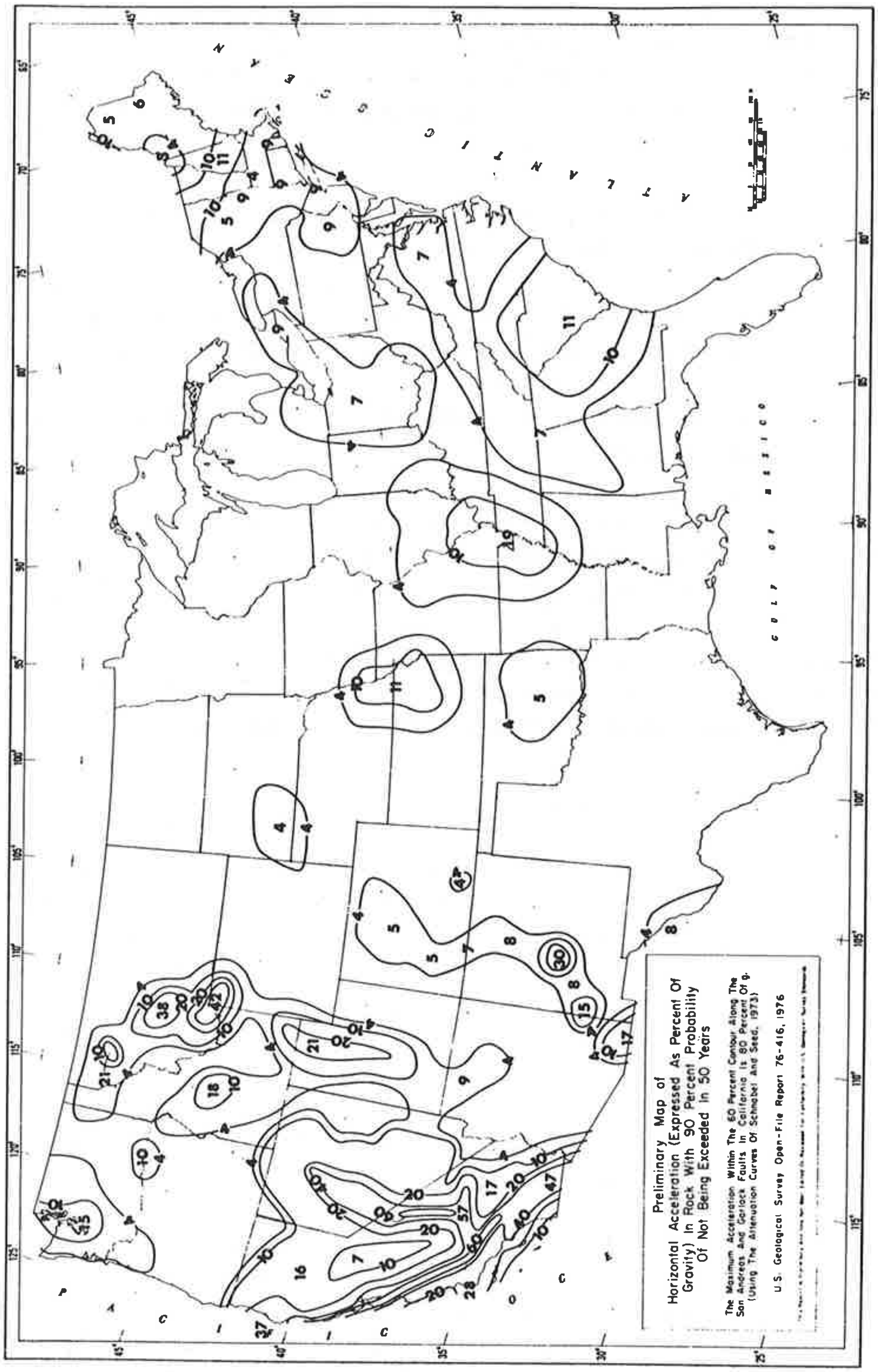


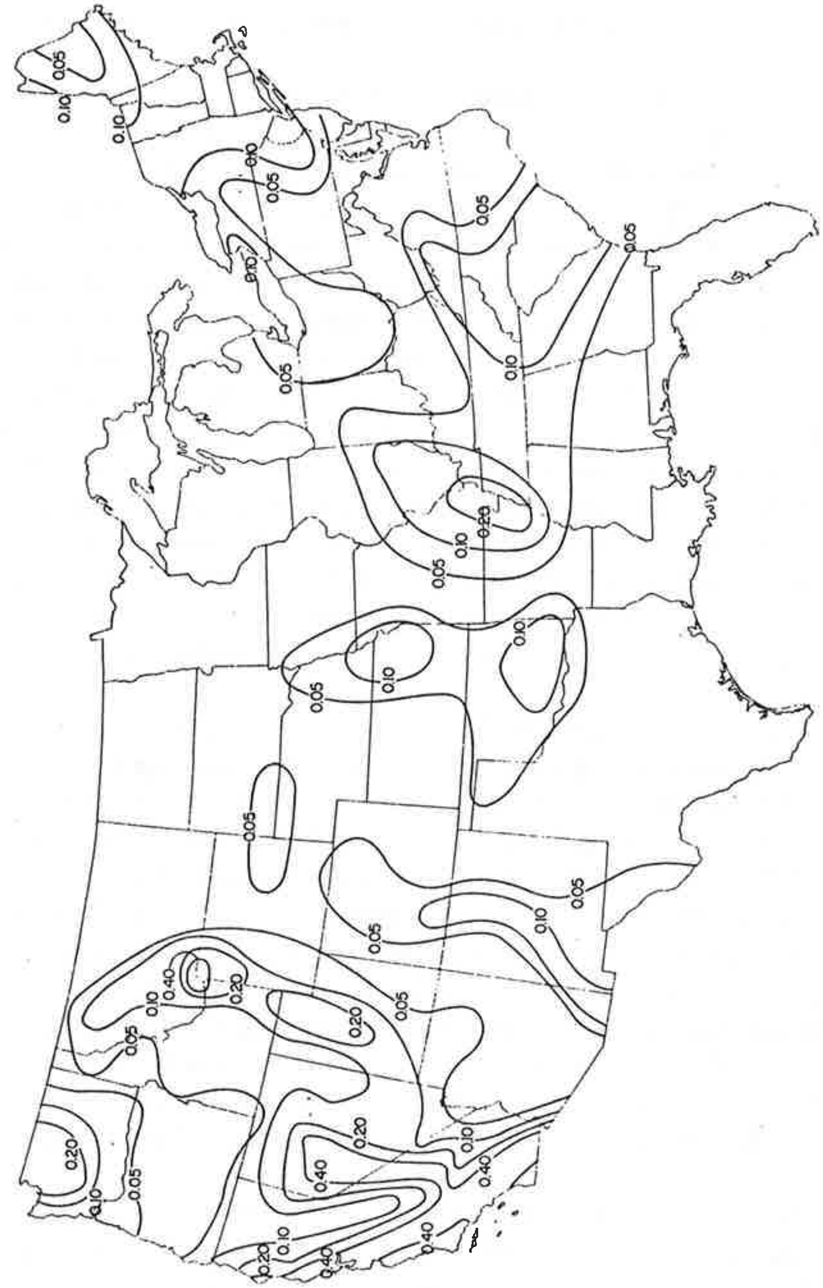
FIGURE A-2 Seismic risk map developed by Algermissen and Perkins.

A contour map for EPA for the contiguous states was developed during the ATC study (1978) that led to development of the preliminary version of these provisions and is given in Figure A-3. (Note that the numbers on the contours in Figure A-3 are values of EPA in units of acceleration of gravity; this map was later converted into map 1 of the *Provisions* 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, 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 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 A-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 then available for purposes of design.

The most significant difference between Figures A-2 and A-3 occurs in the area of highest seismicity in California. Within this region, the Algermissen-Perkins map has contours of 0.6*g*. Whereas, the map for EPA has no values higher than 0.4*g*. There are several reasons for this difference, all of which contributed to a decision to limit EPA to 0.4*g*. 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 0.4*g* will not really bring about more earthquake-resistant construction. Further although there may be locations inside of the 0.4*g* contour where higher values would be appropriate, contouring such small areas would amount to microzoning. In short, the decision to limit the EPA to 0.4*g* was based in part on scientific knowledge and in part on judgment and compromise.

Figure A-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 this information was used to construct maps of EPA that were judged to be consistent with the map for the contiguous 48 states.



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

FIGURE A-3 Contour map for effective peak acceleration (EPA) coefficient, A_a, for the continental United States.

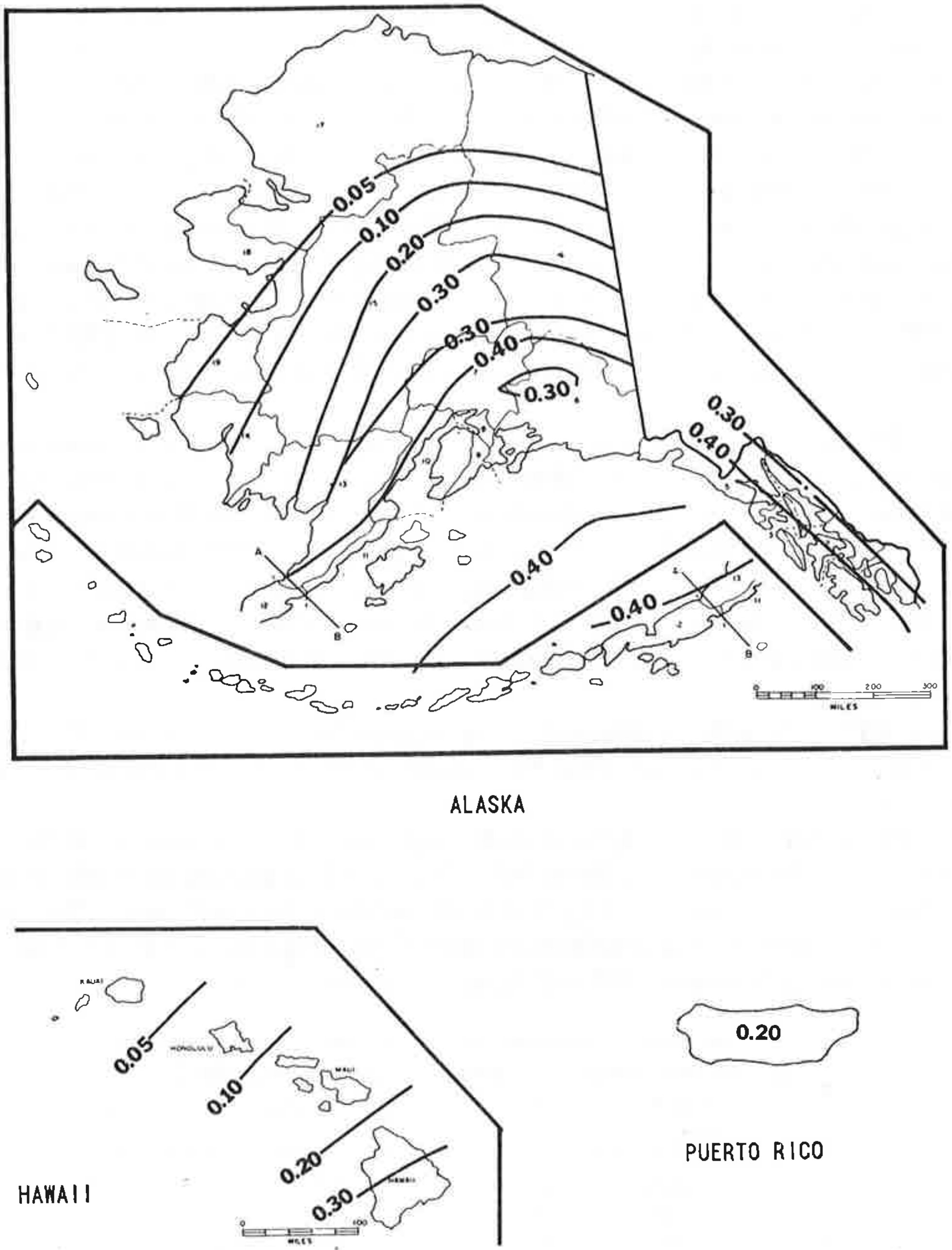


FIGURE A-4 Contour map for effective peak acceleration (EPA) coefficient, A_a, for Alaska, Hawaii, and Puerto Rico.

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); any existing solid geological evidence that this rather short period of history might be misleading was incorporated into the source model. Thus, areas that 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 are 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 actually may have been a larger distant event. These same difficulties apply to the map of EPA, but some geological and seismological studies lead to the EPA being increased in some parts of the country where the historical record alone would have indicated 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 even though current understanding of earthquake occurrences indicates that this assumption is of limited validity. Nevertheless, 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 was available for EPV and, the maps for EPV (Figures A-5 and A-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 (mm/s). For ease in developing the formulas in Sec. 2.3.2, it proved desirable to also express EPV by a dimensionless parameter (A_v) that is an acceleration coefficient. This parameter is referred to as the velocity-related acceleration coefficient. Figures A-5 and A-6 show contours of A_v. The relationship between EPV and A_v is as follows:

Effective Peak Velocity in in./s (mm/s)	Velocity-Related Acceleration Coefficient (A_a)
12 (305)	0.4
6 (152)	0.2
3 (76)	0.1
1.5 (38)	0.05

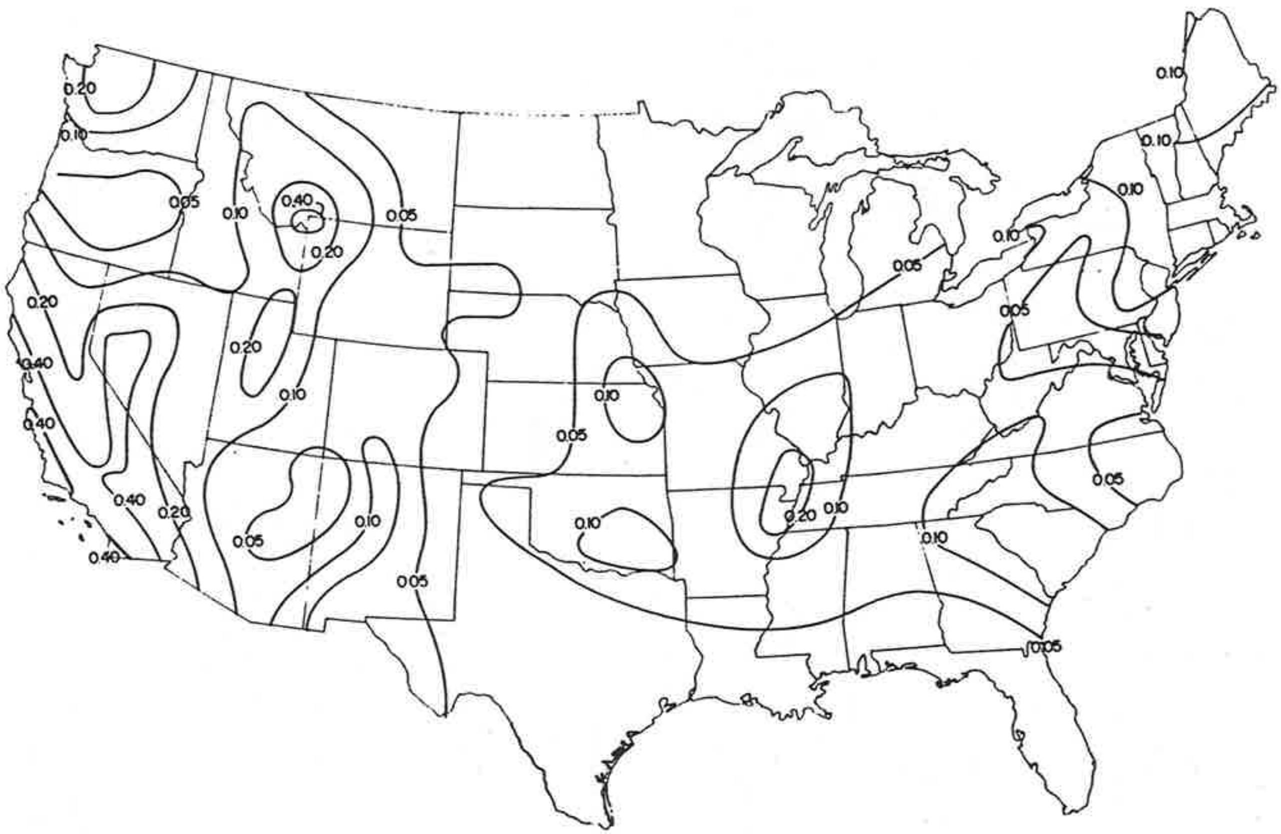


FIGURE A-5 Contour map for effective peak velocity-related acceleration (EPV) coefficient, A_v, for the continental United States.

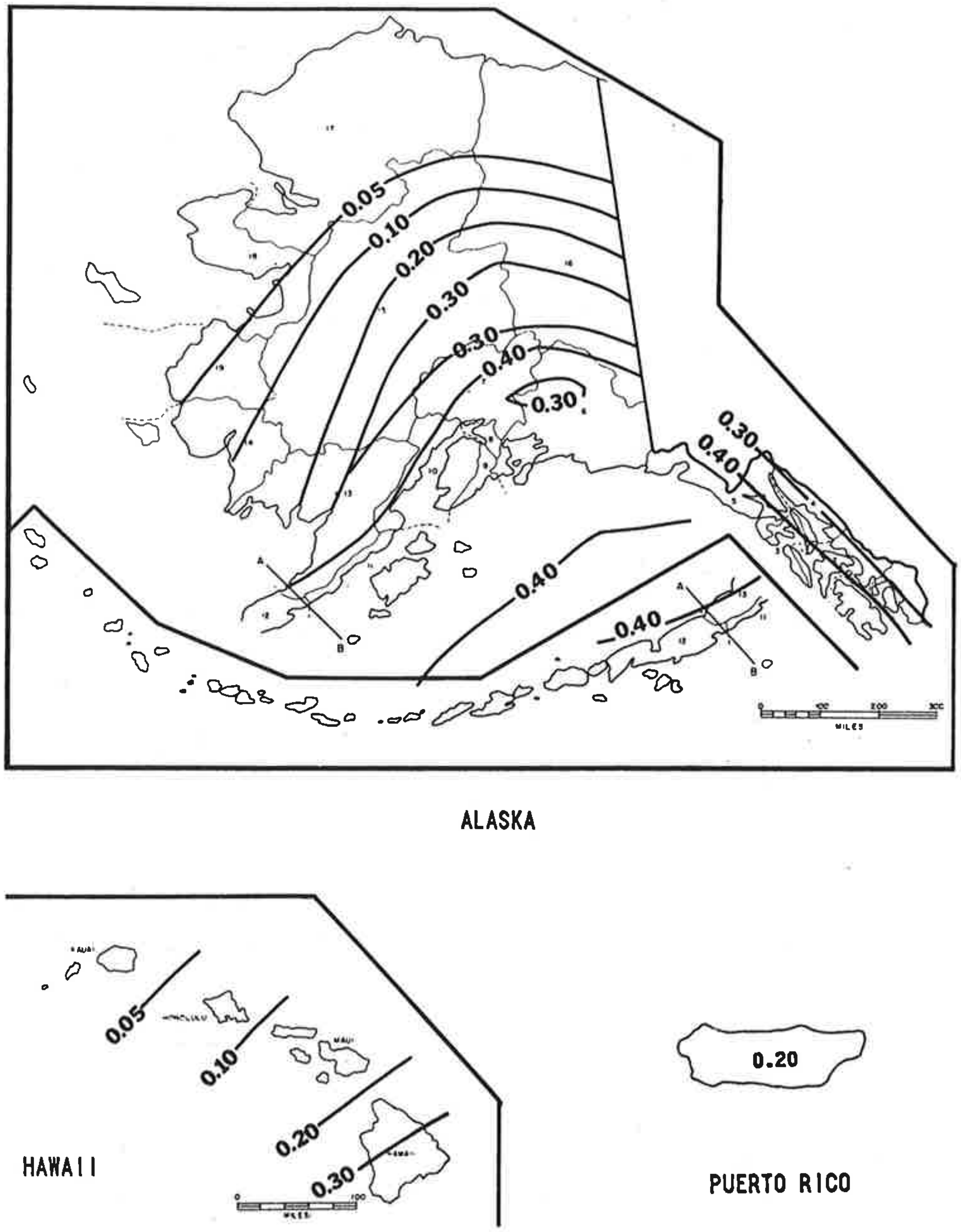


FIGURE A-6 Contour map for effective peak velocity-related acceleration (EPV) coefficient, A_v, for Alaska, Hawaii, and Puerto Rico.

The first step was to assume that the elastic response spectrum for firm ground would apply along the contours for EPA = 0.4*g* in Figure A-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 = 0.4*g*, it is necessary to have EPV = 12 in./s (305 mm/s) to construct this spectrum.

A similar assumption was made for all the peaks of the contour map for EPA (i.e., at all locations where a contour gives the highest EPA in a region). For example, the EPV was set at 3 in./s (76 mm/s) along the contour for EPA = 0.1*g* in the vicinity of the Appalachian Mountains and South Carolina.

A study by McGuire (1975) based on strong motion records in California 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 (129 km). Thus, in the western part of the country, the contours for EPV = 6 in./s (152 mm/s) were located at a distance of about 80 miles (129 km) outside of the contours for EPV = 12 in./s (30 mm/s). Similarly, in Washington and Utah where the highest contour is at 0.2*g*, corresponding to EPV = 6 in./s (152 mm/s), the next contour for EPV = 3 in./s (76 mm/s) was located about 80 miles (129 km) away.

The strong-motion data available to McGuire were inadequate beyond a distance of about 100 miles (161 km). 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 in./s (152 mm/s) centered on southeastern Missouri, the contour for EPV = 3 in./s (76 mm/s) would be about 80 miles (129 km) away and the contour for EPV = 1.5 in./s (38 mm/s) would be 160 miles (258 km) beyond that for 3 in./s (76 mm/s).

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 in./s (76 mm/s) in southcentral Illinois was determined by the contour for EPA = 0.1*g* rather than by distance from the contour for EPV = 6 in./s (152mm/s).

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.

Maps for EPV in the northern half of California were prepared using methods similar to those used for peak acceleration and they gave results consistent with the contours on Figure A-5. Thus, maps in Figures A-5 and A-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. However, given the present state of knowledge, this probability cannot be estimated precisely and, 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 (i.e., 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 A-7, which is based on information supplied by Algermissen and Perkins from their study, identifies probabilities of not being exceeded with other levels of EPA. For example, consider a location on the contour for EPA = $0.2g$ in Figure A-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 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 or at least in the general range of 80 to 95 percent.

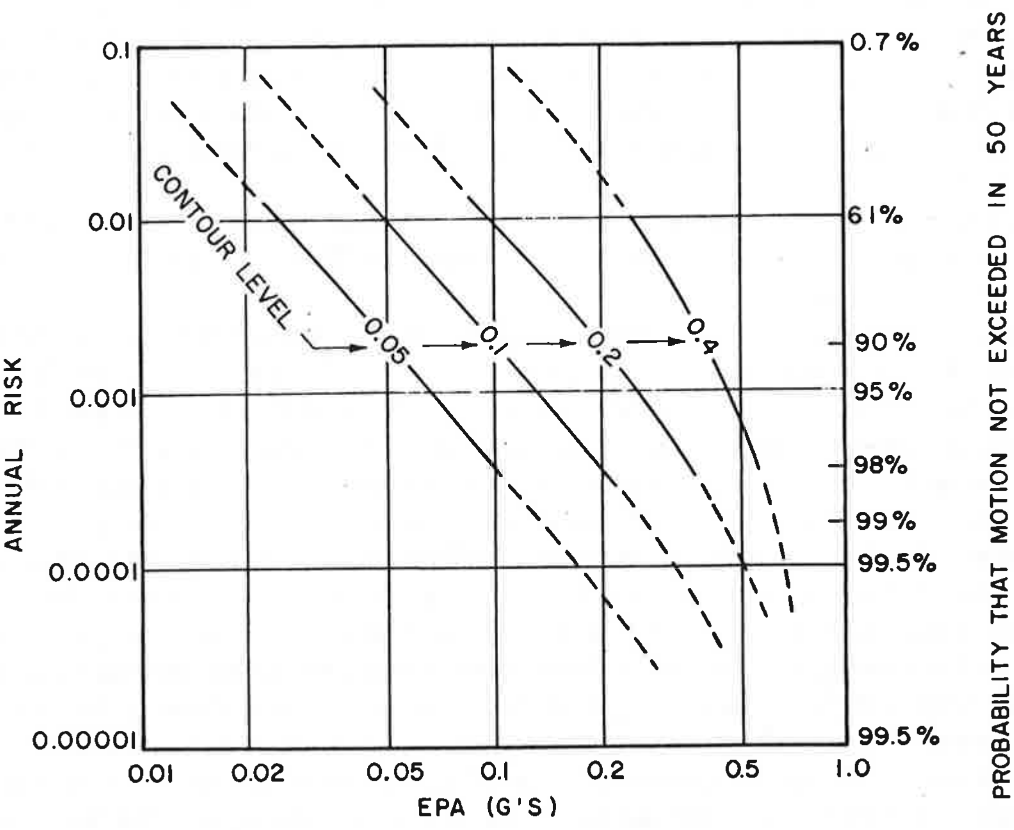


FIGURE A-7 Annual risk of exceeding various EPAs for locations on the indicated contours of EPA in Figure A-3.

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

1. The characteristics of the soil deposits underlying the proposed site,

2. The magnitude of the earthquake producing the design ground motions,

3. The source mechanism of the earthquake producing the ground motions, and

4. 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 permit a general characterization of 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, 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, considers only the effects of site conditions and the distance from the seismic source zone. At such time 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.

County-by-County Maps: Exposure to seismic hazard decreases as the distance from an active seismic region increases, and it was in recognition of this simple premise that the broad uniform zoning being considered in the early 1970s was abandoned during the ATC project leading to the preliminary version of the *Provisions*. Instead, it was initially decided that seismic zoning should be on the basis of the contours shown on Figures A-3 through A-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 that used "Map Areas" with specified values of A_a or A_v with boundaries along 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 in use, both continue to be published with the *Provisions*, and Maps 3 through 12 are in contour form printed over a county line background.

Either the county-by-county seismic design regionalization maps presented in Chapter 1 of the *Provisions* as Maps 1 and 2 or the contour maps in Maps 3 and 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, but in developing the county-by-county maps, intermediate contours were drawn for coefficient values of 0.3 and 0.15 (not shown on Figures A-3 and A-5). The procedure of assigning the same value throughout a county obviously produces discontinuities in some areas of the map; therefore, 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.

The values of the coefficients A_a or A_v associated with Map Areas, presented in Table 1.4.1.1 of the *Provisions*, 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.

REFERENCES AND BIBLIOGRAPHY

Algermissen, S. T., and D. M. Perkins. 1976. *A Probabilistic Estimate of Maximum Acceleration in Rock in the Contiguous United States,* USGS Open File Report 76-416. Reston, Virginia: U.S. Geological Survey.

Algermissen, S. T., and D. M. Perkins. 1972. "A Technique of Seismic Risk Zoning, General Considerations and Parameters." In *Proceedings of the Microzonation Conference*, pp. 865-877. Seattle: University of Washington.

Algermissen, S. T., E. V. Leyendecker, G. A. Bollinger, N. C. Donovan, J. E. Ebel, W. B. Joyner, R. W. Luft, and J. P. Singh. 1991. "Probabilistic Ground-Motion Hazard Maps of Response Spectral Ordinates for the United States." Paper for the Fourth International Conference on Seismic Zonation, Stanford University, Stanford, California.

Algermissen, S. T., D. M. Perkins, P. C. Tenhaus, S. L. Hanson, and B. L. Bender. 1990. *Probabilistic Earthquake Acceleration and Velocity Maps for the United States and Puerto Rico*, U.S. Geological Survey Map MF-2120. Reston, Virginia: USGS.

Algermissen, S. T., D. M. Perkins,. P. C. Thenhaus, S. L. Hanson, and B. L. Bender. 1982. *Probabilistic Estimates of Acceleration and Gravity in Rock in the Contiguous United States*, USGS Open File Report 82-1033. Denver: USGS.

Applied Technology Council. 1974. *An Evaluation of a Response Spectrum Approach to Seismic Design of Buildings,* Report ATC-2. San Francisco, California: Applied Technology Council.

Applied Technology Council. 1978. *Tentative Provisions for the Development of Seismic Regulations for Buildings,* Report ATC 3-06. San Francisco, California: Applied Technology Council.

Blume, J. A., R. L. Sharpe, and J. Dalal. 1973. *Recommendations for Shape of Earthquake Response Spectra,* Report prepared for the Directorate of Licensing. Washington, D.C.: U.S. Atomic Energy Commission.

Bollinger, G. 1976. "Reinterpretation of the Intensity Data for the 1886 Charleston, South Carolina Earthquake." *Bulletin of the Seismological Society of America.*

Cornell, C. A. 1968. "Engineering Seismic Risk Analysis." *Bulletin of the Seismological Society of America* 58:1583-1606.

Culver, C. G., H. S. Lew, G. C. Hart, and C. W. Pinkham. 1975. *Natural Hazards Evaluation of Existing Buildings,* Report BSS 61. Washington, D.C.: U.S. Department of Commerce, National Bureau of Standards.

Foss, J. W. 1977. "Protecting Communication Equipment Against Earthquakes." In *Proceedings of U.S.-Japan Seminar on Earthquake Engineering Research with Emphasis on Lifeline Systems.*

Mark, R. K. 1977. "Application of Linear Statistical Models of Earthquake Magnitude Versus Fault Length in Estimating Maximum Expectable Earthquakes." *Geology* 5:464-466.

McGuire, R. K. 1975. *Seismic Structural Response Risk Analysis, Incorporating Peak Response Progressions on Earthquake Magnitude and Distance,* Report R74-51. Cambridge: Massachusetts Institute of Technology, Department of Civil Engineering.

McGuire, R. K., and K. M. Sedlock. 1981. "Statistical Uncertainties in Seismic Hazard Evaluations in the United States." *Bulletin of the Seismological Society of America* 71(4):1287-1308.

Mohraz, B. 1976. "A Study of Earthquake Response Spectra for Different Geological Conditions." *Bulletin of the Seismological Society of America* 66 (3):915-935.

National Bureau of Standards and Building Seismic Safety Council. 1982. *Amendments to ATC 3-06, Tentative Provisions for the Development of Seismic Regulations for Buildings, for Use in Trial Designs.* Washington, D.C.: U.S. Department of Commerce.

Newmark, N. M. 1973. *A Study of Vertical and Horizontal Spectra,* Report WASH -1255. Washington, D.C.: U.S. Atomic Energy Commission, Directorate of Licensing.

Newmark, N. M., and W. J. Hall. 1969. "Seismic Design Criteria for Nuclear Reactor Facilities." In *Proceedings of the Fourth World Conference on Earthquake Engineering.*

Perkins, David M., Samuel T. Harding, and Steven C. Harmsen. 1988. "A Provisional Peak Velocity Attenuation Compatible with the Schnabel-Seed Acceleration Attenuation." In *Seismic Hazard Studies in the United States,* edited by S. T. Algermissen, USGS Professional Paper. Denver, Colorado: USGS.

Thenhaus, P. C., J. I. Ziony, W. H. Diment, M. G. Hopper, D. M. Perkins, S. L. Hanson, and S. T. Algermissen. 1985. "Probabilistic Estimates of Maximum Seismic Horizontal Ground Acceleration on Rock in Alaska and the Adjacent Continental Shelf." *Earthquake Spectra* 1(2):286-306.

Wiggins, J. H., J. D. Krohn, and J. E. Slosson. 1977. *Earthquake Hazards Analysis: Expected Loss Assessment of Earthquake--Expansive Soils--Landslide*, Technical Report 76-1246-1. Washington, D.C.: National Science Foundation.

Appendix B

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:

1. 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.)

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

3. 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, Phoenix, New York, Chicago, Ft. Worth, Charleston, and St. Louis).

These case studies were developed on the basis of a 1983-84 BSSC trial design program conducted 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, 1988, and 1991 Editions and again during the updating process resulting in this 1994 Edition. Thus, as noted by the BSSC (1984): "Some buildings showing high cost impacts [would] be significantly affected by new amendments...that should tend to reduce the impact." (See Appendix E of this *Commentary* volume for a detailed description of the trial design program.)

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

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

1. 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.

2. 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 B1-1 presents an overview of the construction cost impacts by type of building occupancy. The third column in Table B1-1 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 B-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 B-1 and B-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 B-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 B-2 presents data similar to that in Table B-1 but for each city grouped according to whether the city that had a seismic building code or not. As expected, the average estimated change in the structural cost is considerably higher (more than twice as high) for those cities that had 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 having seismic provisions in their local codes.

TABLE B-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 in 1983-84

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%.

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 no analyses of the cost effect of the 1985, 1988, and 1991 Editions of the *NEHRP Recommended Provisions* have 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. 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.

REFERENCES AND BIBLIOGRAPHY

Building Seismic Safety Council. 1984. *BSSC Program on Improved Seismic Safety Provisions*. 3 volumes. Washington, D.C.: Building Seismic Safety Council.

Dodge Construction System Costs. 1984. New York: McGraw-Hill.

Starr, C. 1969. "Social Benefit vs. Technical Risk." *Science* 165 (September):1232-1238.

Structural Engineers Association of California. 1970. *Report of the Ad Hoc Committee on Cost of Design for Earthquakes.* Washington, D.C.: Office of Science and Technology, Task Force on Earthquake Hazard Reduction.

Weber, Stephen F. 1985. "Cost Impact of the NEHRP Recommended Provisions on the Design and Construction of New Buildings." In *Societal Implications: Selected Readings*, prepared by the Building Seismic Safety Council, FEMA Earthquake Hazards Reduction Series 14. Washington D.C.: Federal Emergency Management Agency.

Wiggins, J. H. 1975. "Paper in Risk-Benefit Methodology and Applications." In *Some Papers Presented at Engineering Foundation Workshop,* edited by David Okrent. Report UCLA-ENG-7598. Asilomar, California: UCLA.

Appendix C

SEISMIC RISK

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 from earthquake effects 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, 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 indirect adverse effects on 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 would 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 makes 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 are discussed below.

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 also is 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.

$$f = P[F|a] \frac{d\gamma}{da} da \tag{C-1}$$

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 C-1). 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 A-7.

Estimated Performance of Buildings Designed According to these 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, 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 C-2 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 C-1 and C-2 has been used as input to Eq. C-1 to compute failure probabilities for four buildings: one located on the contour in Figure C-2b for 0.4*g* and designed for that EPA, one on the contour for 0.2*g* and designed for that EPA, and likewise for buildings located on the 0.10*g* and 0.05*g* contours. In each case, several different assumptions were made as to how the solid line in Figures C-1 and C-2 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 0.05*g* and 0.10*g*, the result is influenced strongly by the way in which the curves of Figures C-1 and C-2 are extrapolated to larger values of EPA or EPV. On the other hand, the results for a building located on the contour for 0.4*g* are influenced strongly by the extrapolations to smaller values of EPA or EPV.

Table C-1 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. However, 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. 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 C-1. 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*. 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 C-1 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.

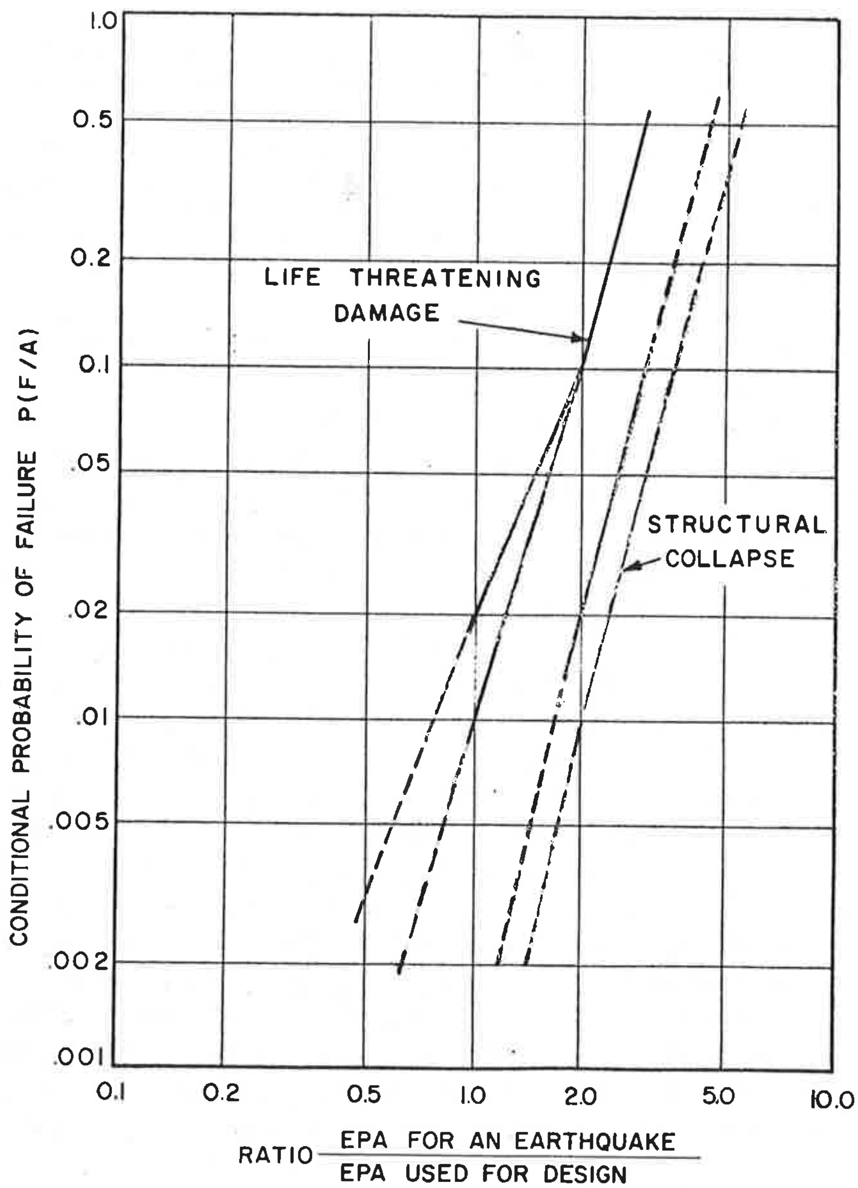


FIGURE C-1 Probability of failure as a function of actual earthquake relative to design earthquake.

TABLE C-1
Probability of Not Having Any Failures During a 50-Year Period (in percent)

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

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), and 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.

There also 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, Figure C-2b summarizes data for the probability of man-made and natural disasters causing greater than various numbers of fatalities. Obviously, these data reflect past experience 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 Figure C-2b. 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.

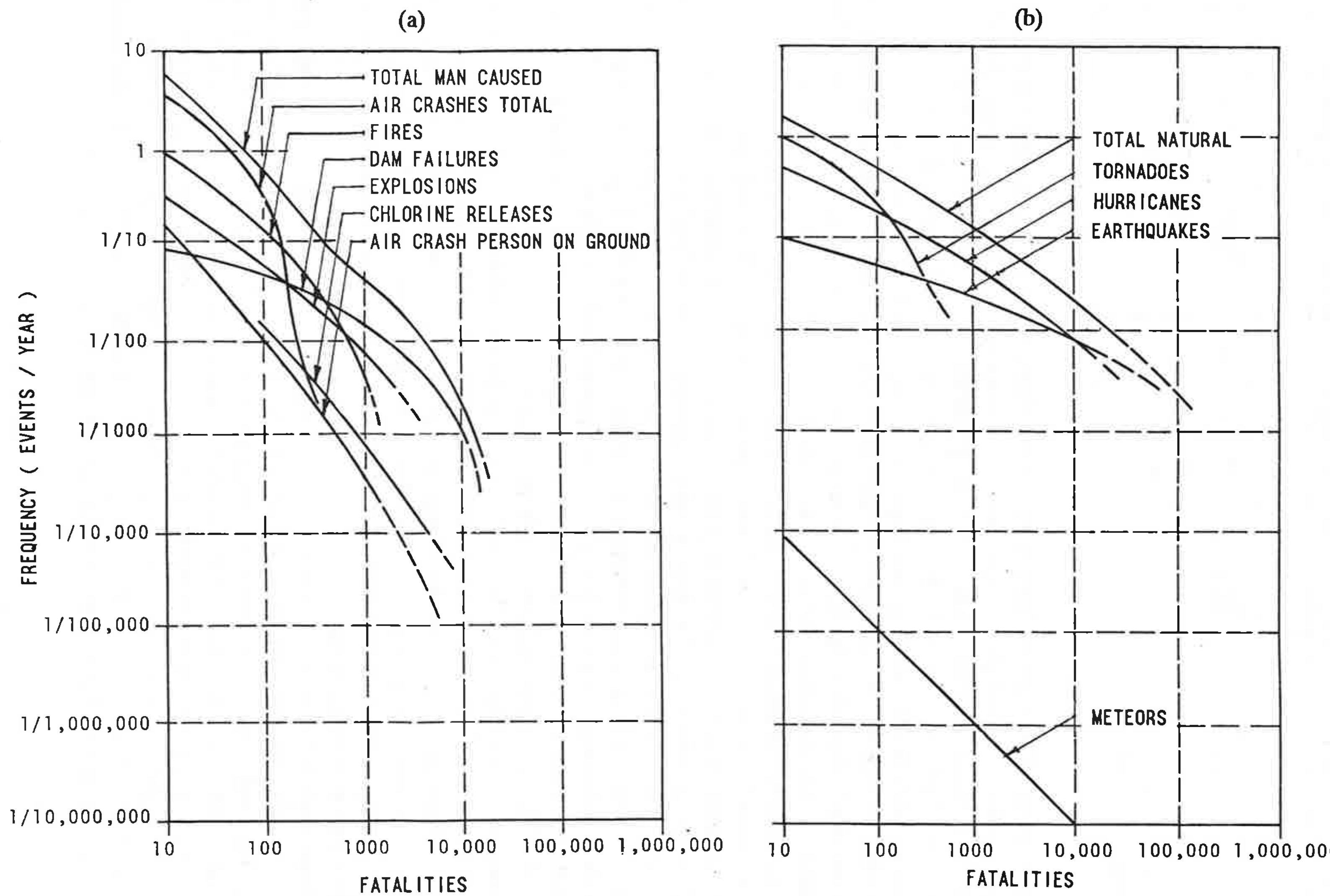


FIGURE C-2 Fatalities due to (a) man-caused failures and (b) natural disasters (U.S. Nuclear Regulatory Commission, 1976).

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.

Other Viewpoints: The technical approaches described above may be helpful to making decisions concerning whether or not the level of risk implicit in a proposed course of action is acceptable; however, these approaches do not by themselves dictate what should be done. This is the purview of legislative, administrative, and judicial bodies.

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.

REFERENCES AND BIBLIOGRAPHY

Starr, C. 1969. "Social Benefit vs. Technical Risk." *Science* 165 (September):1232-1238.

Wiggins, J. H. 1975. "Paper in Risk-Benefit Methodology and Applications." In *Some Papers Presented at Engineering Foundation Workshop,* edited by David Okrent. Report UCLA-ENG-7598. Asilomar, California: UCLA.

U.S. Nuclear Regulatory Commission. 1976. *Reactor Safety Study,* Report WASH-1400. Washington, D.C.: U.S. Nuclear Regulatory Commission.

Appendix D

PRELIMINARY SITE RESPONSE RECOMMENDATIONS: MEMORANDUM FROM EDWARD RINNE AND RICARDO DOBRY TO ROLAND SHARPE, DECEMBER 11, 1992

This summarizes recommendations generated at the NCEER/SEAOC/BSSC Site Response Workshop at USC on November 18-20, 1992. The preparation of site response code proposals and subsequent discussion and consensus at the workshop among the organizing committee and TS-3 have resulted in a recommended revised site classification system and a simplified method of constructing free-field, elastic response spectra, as summarized in Attachment I and subsequent Tables 1.1, 1.2, and 1.3.

Attachment I sketches the proposed method to construct the free-field response spectrum for a given site class and specified values of A_a and Av, assumed known for Site Class A (Rock). The site classification system of Table 1.1 recognizes the primary importance of the types of soil materials and especially their shear wave velocities in the top 100 feet of the site profile, as well as the need to consider the difference in response of rocks of different stiffness. The low- and high-period spectral site coefficients F_a and F_v in Tables 1.2 and 1.3 are functions of both site class and level of shaking.

The exact form and the details of this enclosed material should be considered preliminary at this stage, and work by the organizing committee and TS-3 in refining them continues. The coefficients F_a and F_v will have to be redefined if A_a and A_v are replaced by other mapped spectral parameters, or if these mapped spectral parameters are specified for site conditions other than rock. Table 1.1 is still quite sketchy and does not include yet enough geotechnical descriptions and logical statements to make it a workable code tool; also, site class (E), for which site specific analyses are mandated, may require specification of "fall back," high, conservative values of F_a and F_v for users who don't want - or can't - conduct these site-specific studies. However, the basic approach contained in Attachment I and the three tables, the number of site classes and their basic definitions in Table 1.1, and the values of F_a and F_v in Tables 1.2 and 1.3, will not change or will change only slightly, and thus can be used with confidence at this stage as a starting point for your work. To help the work of TS-2 in understanding Table 1.1, we have indicated on it by hand the general correspondence between the new site classes A_0 ... D and the old site categories S_1 ... S_4. It is the intent of the recommended elastic free-field spectra embodied in this methodology to form the basis for developing appropriate site coefficients for static lateral force analyses for buildings that better recognize the influence of various site conditions. In this regard, the TS-3 recommends

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consensus proposal contained in Attachment I and Tables 1.1 to 1.3. It must be emphasized that the proposed values of F_a and F_v in Tables 1.2 and 1.3 for low $A_a \simeq 0.1$ g and $A_v \simeq 0.1$ g are firmly grounded on empirical results, especially from the Loma Prieta earthquake. At these low levels of rock acceleration, the values of F_a and F_v obtained from the empirical and analytical studies agree well, and this provided a calibration point for the analytical techniques used (mostly 1D equivalent linear and nonlinear codes). On the other hand, the values of F_a and F_v at high A_a and A_v such as 0.4 g are mostly based on these calibrated analytical techniques.

The proposed methodology for constructing response spectra is based on the current acceleration and velocity-based effective peak accelerations (A_a and A_v) presented on Maps 3 and 4 of the 1991 NEHRP Provisions, but the method could be easily modified for other spectral maps which may eventually work their way into the provisions. The method to construct response spectra contained in Attachment I is the same as presented to the workshop on November 20th. It should be noted that these spectra are intended to cover the period range of about 0.2 seconds to 3.0 seconds, or the portion of the spectra controlled by nearly constant spectral acceleration and velocity in the classic Newmark-Hall method. The method does not address the period range between 0 and about 0.2 seconds, and thus cannot be used to amplify peak acceleration or other high frequency spectral values.

The site classes shown on Table 1.1 are unchanged from those presented at the workshop. The classifications in terms of shear wave velocity values were agreed upon at the workshop; however, the geotechnical committee is in the process of refining the descriptive geotechnical terms and definitions and providing guidance to practicing engineers on the correlation of shear wave velocity to common site investigation data, e.g., standard penetration, CPT, undrained shear strength, water content, etc. This ongoing effort should not affect the progress of developing revised lateral coefficients, but can be conducted in parallel.

While the F_a and F_v values on Tables 1.2 and 1.3 appear to represent a significant increase in the current S factors, this increase is less significant when compared to the relative spectral values for various conditions presently in the UBC and the NEHRP Commentary. Furthermore, a comparison of the method for site Class C conditions to the Chapter 1 Appendix Spectral Maps appears to be in the same ballpark.

Lastly, some commentary on the risk levels and uncertainties relative to the recommended method would seem appropriate. The method generally applies to the 90% probability of non-exceedance in 50 years that forms the basis for the present code. It does not, however, incorporate the uniform risk approach used in the Spectral Maps. The areas of uncertainty and the method used in dealing with it can be summarized as follows:

1. Use of the current A_a and A_v map values should recognize that these maps were prepared over 20 years ago using historical data available at the time and using mean attenuation relationships that do not account for variability. In addition, the values are truncated to 0.40 g maximum. These effects may lead to unconservative results particularly in high seismicity zones near active faults. Site specific studies are recommended within 10 km of an active fault to better evaluate near fault conditions.

2. The F_a values in Table 1.2 generally represent mean values based on the limitations of the studies. There is considerable uncertainty at higher rock input motions due to limited empirical data and analyses of same. In addition, the degree of uncertainty in these values is not incorporated into the method which could have either over or under conservative implications depending on the site.

3. The F_v values in Table 1.3 generally represent mean plus about one sigma because the actual value is highly variable depending on the specific period being considered, site conditions and input motion. In the period range of highest site amplification (typically associated with resonance near the site period), the proposed F_v values are well below the mean, while at periods of relatively low amplification, they are much higher than the mean. The selection of mean plus sigma was made to provide better protection for the high amplification period range although it is still below the mean based on both analytical and empirical results near the site period.

While this is a fairly brief synopsis of a very substantial effort, we hope that the main information needed by the design community has been covered sufficiently to get started in the process of developing appropriate lateral force coefficients. Again we on TS-3 look forward to working with TS-2 and others in this effort.

cc: Jim Smith, BSSC
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TS-3 Committee
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Attachment I:

Method to Construct Site-Specific Response Spectra

(Free-field, Elastic, Smoothed, 5% Damping)

1. Select A_a and A_v from Maps 3 and 4.

2. Select appropriate Site Class (or "category") from Table 1.1.

3. Obtain the corresponding values of F_a and F_v from Tables 1.2 and 1.3, respectively.

4. Compute the (short period) constant spectral acceleration portion of the response spectrum as

$$S_A = (2.5) \times F_a \times A_a \qquad \text{[Eq. 1.1]}$$

5. Compute the (longer period) constant velocity portion of the spectrum [S_A decreases as $1/T$ where T = Period] as:

$$S_A = F_v \times A_v \times \left(\frac{1}{T}\right) \qquad \text{[Eq. 1.2]}$$

noting that at $T = 1.0$ seconds:

$$S_{A(1.0)} = F_v \times A_v$$

6. At each period (T), the elastic response spectrum is the lesser of the two values from Equations 1.1 and 1.2, so that Equation 1.1 defines S_A in the low-period range, and Equation 1.2 defines S_A in the higher period range. The period at which the transition from Eq. 1.1 to 1.2 occurs varies, as a function of (a) site class, (b) A_a and (c) A_v.

TABLE 1.1: Preliminary Site Classification for Seismic Site Response

"Old" Site Category	Site Class	Site Class Name/ Generic Description[5]	Site Class Definition[1 3 4 5]
S_1	A_0	Hard Rock	$\overline{V}_s > 5{,}000$ ft/sec
	A	Rock	2,500 ft/sec $< \overline{V}_s <$ 5,000 ft/sec
S_1 and S_2	B	Hard and/or stiff/very stiff soils; most gravels	1,200 ft/sec $< \overline{V}_s <$ 2,500 ft/sec
	C	Sands, silts and/or stiff/very stiff clays, some gravels	600 ft/sec $< \overline{V}_s <$ 1,200 ft/sec
S_3 and S_4	D	Profile containing a small-to-moderate total thickness H of soft/ medium stiff clay	$\overline{V}_s <$ 600 ft/sec and/or 10 ft $<$ H $<$ 50 ft
	D_2	Profile containing a large total thickness H of soft/medium stiff clay	$\overline{V}_s <$ 600 ft/sec and/or 50 ft $<$ H $<$ 120 ft
	(E)[2 6]	(E_1) - Soils Vulnerable to Potential Failure or Collapse Under Seismic Loading: [Liquefiable Soils, Quick and Highly Sensitive Clays, Collapsible Weakly-Cemented Soils, etc.] (E_2) - Peats and/or Highly Organic Clays: [H > 10 ft of peat and/or highly organic clay] (E_3) - Very High Plasticity Clays: [H > 25 ft with PI > 75%] (E_4) - Very Thick "Soft/Medium Stiff Clays" [H > 120 ft]	

Notes for Table 1.1:

1. A site is classified in principle on the basis of the average shear wave velocity $\overline{V}_s$ as defined in Note 3, modified as specified by layer thickness criteria.

2. These conditions require site-specific studies. The presence of one or more of the conditions listed (E_1 through E_5) takes precedence over alternate possible classifications; if a site could be classified as both: (a) one of the six site classes (A_0, A, B, C. D, or D_2), and (b) also as one or more of the "E" classes, then the "E" classification takes precedence. If the "E" classification is the result of potential for liquefaction or other forms of ground failure (E_1) and if this potential is eliminated by remedial works, then the site may be reclassified.

3. $\overline{V}_s$ is a weighted-average shear wave velocity over the top 100 feet immediately below the ground surface (as it will exist upon completion of the proposed project). The weighting system employed is designed to implicitly account for the effects of layer (or stratum) thicknesses by assigning increased emphasis to layers of lower

shear stiffness (or lower shear wave velocity; V_s). $\overline{V}_s$ is calculated by summing the vertical shear wave travel times (t_s) within each sublayer or sub-stratum within the top 100 feet.

$$\text{Within the } i^{th} \text{ sublayer: } t_{s,i} = \frac{h_i}{v_{s,i}}$$

where: h = layer thickness, and
$v_{s,i}$ = shear wave velocity within the sublayer.

The total vertical travel time over the top 100 ft ($t_{s,100}$) is then calculated by summing the individual sublayer travel times as

$$t_{s,100} = \sum_{i=1}^{100\ ft} t_{s,i}$$

Finally, the weighted-average shear wave velocity ($\overline{V}$ over the top 100 feet is established as the travel distance (100 ft) divided by the travel time:

$$\overline{V}_s = \frac{100\ ft}{t_{s,100}}$$

4. Vertical shear wave velocities can either be measured directly (in-situ), or can be estimated based on suitable empirical correlations developed for similar types of materials. Many such correlations exist, and are sufficiently accurate as to be useful for this purpose. A discussion of methods for measuring and/or otherwise evaluating V_s will be included in the Commentary.

5. Column 2 of the table contains a very general description of the types of soils associated with the corresponding range of $\overline{V}_s$ listed in Column 3. The final version of the table will add a Column 4, not included here, with much more detailed geotechnical descriptions and ranges of geotechnical parameters such as SPT and Su, to be used to classify the site in the absence of shear wave velocity information. (SPT = Standard Penetration Test; Su = undrained shear strength of clay)

6. PI = Plasticity Index (%)
   = LL (%) - PL (%)

   where

   LL = Liquid Limit and
   PL = Plastic Limit

   (Atterberg Limits)

Table 1.2: Values of F_a as a Function of Site Conditions and Shaking Intensity

Site Class ↓ / Shaking Intensity →	$A_a = 0.1$ g	$A_a = 0.2$ g	$A_a = 0.3$ g	$A_a = 0.4$ g	$A_a = 0.5$ g
(A_o)	0.8	0.8	0.8	0.8	0.8
A	1.0	1.0	1.0	1.0	1.0
B	1.2	1.2	1.1	1.0	1.0
C	1.6	1.4	1.2	1.1	1.0
D_1	2.5	1.7	1.2	0.9	(--)[1]
D_2	2.0	1.6	1.2	0.9	(--)[1]
(E)	(--)[1]	(--)[1]	(--)[1]	(--)[1]	(--)[1]

[1] Site-specific geotechnical investigations and dynamic site response analyses should be performed.

Table 1.3: Values of F_v as a Function of Site Conditions and Shaking Intensity

Site Class ↓ / Shaking Intensity →	$A_v = 0.1$ g	$A_v = 0.2$ g	$A_v = 0.3$ g	$A_v = 0.4$ g	$A_v = 0.5$ g
(A_o)	0.8	0.8	0.8	0.8	0.8
A	1.0	1.0	1.0	1.0	1.0
B	1.7	1.6	1.5	1.4	1.3
C	2.4	2.0	1.8	1.6	1.5
D_1	3.5	3.2	2.8	2.4	(--)[2]
D	3.5	3.2	2.8	2.4	(--)[2]
(E)	(--)[2]	(--)[2]	(--)[2]	(--)[2]	(--)[2]

[2] Site-specific geotechnical investigations and dynamic site response analyses should be performed.

Appendix E

THE COUNCIL AND ITS PURPOSE

The Building Seismic Safety Council (BSSC) was established in 1979 under the auspices of the National Institute of Building Sciences as an entirely new type of instrument for dealing with the complex regulatory, 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 issues related to the seismic safety of the built environment could be resolved and jurisdictional problems overcome through authoritative guidance and assistance backed by a broad consensus.

The BSSC is an independent, voluntary membership body representing a wide variety of building community interests (see page 12 for a current membership list). 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. Thus, the BSSC itself assumes no standards-making or -promulgating role; rather, it advocates that code- and standards-formulation organizations consider BSSC recommendations for inclusion into their documents and standards.

IMPROVING THE SEISMIC SAFETY OF NEW BUILDINGS

The BSSC program directed toward improving the seismic safety of new buildings has been conducted with funding from the Federal Emergency Management Agency (FEMA). It is structured to create and maintain authoritative, technically sound, up-to-date 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 BSSC program began with initiatives taken by the National Science Foundation (NSF). Under an agreement with the National Bureau of Standards (NBS; now NIST, the National Institute for Standards and Technology), *Tentative Provisions for the Development of Seismic Regulations for Buildings* (referred to here 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 the ATC explained that a careful assessment was needed.

Following the issuance of the *Tentative Provisions* in 1978, NBS released a technical note calling for "...systematic analysis of the logic and internal consistency of [the *Tentative Provisions*]" and developed a plan for assessing and implementing seismic design provisions for buildings. This plan called for a thorough review of the *Tentative Provisions* by all interested organizations; the conduct of trial designs to establish the technical validity of the new provisions and to assess their economic impact; the establishment of a mechanism to encourage consideration and adoption of the new provisions by organizations promulgating national standards and model codes; and educational, technical, and administrative assistance to facilitate implementation and enforcement.

During this same period, other significant events occurred. In October 1977, Congress passed the Earthquake Hazards Reduction Act of 1977 (P.L. 95-124) and, in June 1978, the National Earthquake Hazards Reduction Program (NEHRP) was created. Further, FEMA was established as an independent agency to coordinate all emergency management functions at the federal level. Thus, the future disposition of the *Tentative Provisions* and the 1978 NBS plan shifted 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 creation of a mechanism for obtaining broad public and private consensus on both recommended improved building design and construction regulatory provisions and the means to be used in their promulgation. Following a series of meetings between representatives of the original participants in the NSF-sponsored project on seismic design provisions, FEMA, the American Society of Civil Engineers and the National Institute of Building Sciences (NIBS), 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, at which time a charter and organizational rules and procedures were thoroughly debated and agreed upon.

The BSSC provided the mechanism or forum needed to encourage consideration and adoption of the new provisions by the relevant organizations. A joint BSSC-NBS committee was formed to conduct the needed review of the *Tentative Provisions*, which resulted in 198 recommendations for changes. Another joint BSSC-NBS committee developed both the criteria by which the needed trial designs could be evaluated and the specific trial design program plan. Subsequently, a BSSC-NBS Trial Design Overview Committee was created to revise the trial design 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.

Trial Designs

Initially, the BSSC trial designs that were to include the following building types and structural systems:

Building Types
Low-, mid-, and high-rise residential buildings
Mid- and high-rise office buildings
One-story industrial buildings
Two-story commercial buildings

Construction Types and Structural Systems
Lateral load systems
- Shear walls
  - Cast-in-place concrete
  - Precast and prestressed-precast concrete
  - Masonry
  - Plywood on wood studs
- Braced frames--conventional steel
- Unbraced frames
  - Cast-in-place concrete - special and ordinary
  - Steel, both special and ordinary, conventional and pre-engineered

Vertical load systems
- Bearing wall buildings
  - Walls
    - Cast-in-place concrete
    - Precast and prestressed-precast concrete
    - Masonry
    - Plywood on wood studs
  - Floors
    - Concrete slabs, both cast-in-place and precast, ordinary and prestressed
    - Steel joists with decks and slabs
    - Wood framing with plywood decks and lightweight concrete fill
- Framed buildings
  - Cast-in-place concrete flat slabs, waffle slabs, pan joists, and beam and slab systems, both ordinary
    and prestressed
  - Precast concrete, both ordinary and prestressed
  - Steel girder and purlin, beam and joist, and long-span truss systems with decks and slabs
  - Wood framing

It originally was intended that the trial design effort be conducted in two phases and include trial designs for 100 new buildings in 11 major cities but financial limitations required that the program be scaled down. Ultimately, 17 design firms were retained to prepare trial designs for 46 new buildings in 4 cities with medium to high seismic risk (10 in Los Angeles, 4 in Seattle, 6 in Memphis, 6 in Phoenix) and in 5 cities with medium to low seismic risk (3 in Charleston, South Carolina, 4 in Chicago, 3 in Ft. Worth, 7 in New York, and 3 in St. Louis). Alternative designs for six of these buildings also were included.

The firms participating in the trial design program were: ABAM Engineers, Inc.; Alfred Benesch and Company; Allen and Hoshall; Bruce C. Olsen; Datum/Moore Partnership; Ellers, Oakley, Chester, and Rike, Inc.; Enwright Associates, Inc.; Johnson and Nielsen Associates; Klein and Hoffman, Inc.; Magadini-Alagia Associates; Read Jones Christoffersen, Inc.; Robertson, Fowler, and Associates; S. B. Barnes and Associates; Skilling Ward Rogers Barkshire, Inc.; Theiss Engineers, Inc.; Weidlinger Associates; and Wheeler and Gray.

For each of the 52 designs, a set of general specifications was developed, but the responsible design engineering firms were given latitude to ensure that building design parameters were compatible with local construction practice. The designers were not permitted, however, to change the basic structural type 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 some designers from selecting the most economical system.

Each building was designed twice--once according to the amended *Tentative Provisions* and again according to the prevailing local code for the particular location of the design. In this context, 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), partial nonstructural designs (complete enough to assess the cost of the nonstructural portion of the building), and design/construction cost estimates were developed.

This phase of the BSSC program concluded with publication of a draft version of the recommended provisions, the *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings*, an overview of the *Provisions* refinement and trial design efforts, and the design firms' reports.

The 1985 Edition of the *NEHRP Recommended Provisions*

The draft version represented an interim set of provisions pending their balloting by the BSSC member organizations. The first ballot, conducted in accordance with the BSSC Charter, was organized on a chapter-by-chapter basis. As required by BSSC procedures, the ballot 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 explanations received with "yes with reservations" and "no" votes were compiled, and proposals for dealing with them were developed for consideration by the Technical Overview Committee and, subsequently, the BSSC Board of Direction. The draft provisions then were revised to reflect the changes deemed appropriate by the BSSC Board and the revision was submitted to the BSSC membership for balloting again.

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 issues as possible. The 1985 Edition of the *NEHRP Recommended Provisions* then was transmitted to FEMA for publication in December 1985.

During the next three years, a number of documents were published to support and complement the 1985 *NEHRP Recommended Provisions*. They included a guide to application of the *Provisions* in earthquake-resistant building design, a nontechnical explanation of the *Provisions* for the lay reader, and 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.

The 1988 Edition

The need for continuing revision of the *Provisions* had been anticipated since the onset of the BSSC program and the effort to update the 1985 Edition for reissuance in 1988 began in January 1986. During the update effort, nine BSSC Technical Committees (TCs) studied issues concerning seismic risk maps, structural design, foundations, concrete, masonry, steel, wood, architectural/mechanical/electrical systems, and regulatory use. The Technical Committees worked under the general direction of a Technical Management Committee (TMC), which was composed of a representative of each TC as well as additional members identified by the BSSC Board to provide balance.

The TCs and TMC worked throughout 1987 to develop specific proposals for changes needed in the 1985 *Provisions*. In December 1987, the Board reviewed these proposals and decided upon a set of 53 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 unresolved 1985 edition issues.

The balloting was conducted on a proposal-by-proposal basis in February-April 1988. Fifty of the proposals 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 BSSC Board. Other comments were found to be unpersuasive or in need of further study during the next update cycle (to prepare the 1991 *Provisions*). A number of comments persuaded the TMC and Board that a substantial alteration of some balloted proposals was necessary, and it was decided to submit these matters (11 in all) to the BSSC membership for reballot during June-July 1988. Nine of the eleven reballot proposals passed and two failed.

On the basis of the ballot and reballot results, the 1988 *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. Efforts to update the complementary reports published to support the 1985 edition also were initiated. Ultimately, the following publications were updated to reflect the 1988 Edition and reissued by

FEMA: the *Guide to Application of the Provisions*, the handbook discussing societal implications (which was extensively revised and retitled *Seismic Considerations for Communities at Risk*), and several *Seismic Considerations* handbooks (which are described below).

The 1991 Edition

During the effort to produce the 1991 *Provisions*, a Provisions Update Committee (PUC) and 11 Technical Subcommittees addressed seismic hazard maps, structural design criteria and analysis, foundations, cast-in-place and precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, interface with codes and standards, and composite structures. Their work resulted in 58 substantive and 45 editorial proposals for change to the 1988 *Provisions*.

The PUC approved more than 90 percent of the proposals and, in January 1991, the BSSC Board accepted the PUC-approved proposals for balloting by the BSSC member organizations in April-May 1991.

Following the balloting, the PUC considered the comments received with "yes with reservations" and "no" votes and prepared 21 reballot proposals for consideration by the BSSC member organizations. The reballoting was completed in August 1991 with the approval by the BSSC member organizations of 19 of the reballot proposals.

On the basis of the ballot and reballot results, the 1991 *Provisions* was prepared and transmitted to FEMA for publication in September 1991. Reports describing the changes made in the 1988 edition and issues in need of attention in the next update cycle then were prepared.

In August 1992, in response to a request from FEMA, the BSSC initiated an effort to continue its structured information dissemination and instruction/training effort aimed at stimulating widespread use of the *NEHRP Recommended Provisions*. The primary objectives of the effort were to bring the *Guide to Application of the NEHRP Recommended Provisions in Earthquake-Resistant Design*, the *Seismic Considerations* handbooks (described below), *Seismic Considerations for Communities at Risk*, and the *Non-Technical Explanation of the NEHRP Recommended Provisions* into conformance with the the 1991 *NEHRP Recommended Provisions* in a manner reflecting other related developments (e.g., the fact that all three model codes now include requirements based on the *Provisions*) and to bring instructional course materials currently being used in the BSSC seminar series (described below) into conformance with the 1991 *Provisions*.

The 1994 Edition

The effort to structure the 1994 PUC and its technical subcommittees was initiated in late 1991. By early 1992, 12 Technical Subcommittees (TSs) were established to address seismic hazard mapping, loads and analysis criteria, foundations and geotechnical considerations, cast-in-place and precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, interface with codes and standards, and composite steel and concrete structures, and base isolation/energy dissipation.

The TSs worked throughout 1992 and 1993 and, at a December 1994 meeting, the PUC voted to forward 52 proposals to the BSSC Board with its recommendation that they be submitted to the BSSC member organizations for balloting. Three proposals not approved by the PUC also were forwarded to the Board because 20 percent of the PUC members present at the meeting voted to do so. Subsequently, an additional proposal to address needed terminology changes also was developed and forwarded to the Board.

The Board subsequently accepted the PUC-approved proposals; it also accepted one of the proposals submitted under the "20 percent" rule but revised the proposals to be balloted as four separate items. The BSSC member organization balloting of the resulting 57 proposals occurred between March 31 and May 31, 1994, with 42 of the 54 voting member organizations submitting their ballots. The ballot results and comments then were compiled in preparation for a July 1994 PUC comment resolution meeting.

Meeting in July, the PUC reviewed all of the TS chairmen's responses and proposed revisions, voted on whether to accept the responses (some with further modification), and identified 20 substantive changes that would require reballoting. Of the four proposals that failed the ballot, three were withdrawn by the TS chairmen and one was substantially modified and also was accepted for reballoting. The PUC Chairman presented the PUC's recommendations concerning comment resolution and the 21 items for reballot to the BSSC Board of Direction at a meeting later in July. The Board accepted the PUC recommendations except in one case where it deemed comments to be persuasive and made an additional substantive change to be reballoted by the BSSC member organizations.

The second ballot package was mailed to the BSSC member organizations on September 9 and ballots were to be submitted by October 11. The PUC then assessed the ballot results and made its recommendations to the BSSC Board on November 1. One needed revision identified later was considered by the PUC Executive Committee in December. The copy of the 1994 Edition of the *Provisions* as well as the summary of the differences between the 1991 and 1994 Editions was prepared for final review by the PUC. The camera-ready copy was delivered to FEMA in March 1995.

1997 Update Effort

On September 29, 1994, NIBS entered into a contract with FEMA for initiation of the 39-month BSSC 1997 *Provisions* update effort. Late in 1994, the BSSC member organization representatives and alternate representatives and the BSSC Board of Direction were asked to identify individuals to serve on the 30- to 40-member 1997 PUC and its TSs. Many of the individuals who served on the 1994 update committees also were asked to consider serving again during the 1997 effort to maintain needed continuity. Committee member selection criteria ensures that, insofar as practicable, the update committees are balanced in terms of representation of discipline, geographic area, and interest group (design professionals, regulators, suppliers, consumers, academics, and researchers).

The 1997 PUC was expected to be fully constituted early in 1995. It was anticipated that at least 12 PUC Technical Subcommittees would be required as follows: design criteria and analysis, foundations and geotechnical considerations, cast-in-place/precast concrete structures, masonry structures, steel structures, wood structures, mechanical-electrical systems and building equipment and architectural elements, quality assurance, interface with codes and standards, composite steel and concrete structures, energy dissipation and base isolation, and nonbuilding structures.

As part of the update effort, the BSSC plans to develop a more rational seismic design procedure for use by engineers and architects for inclusion in the 1997 *NEHRP Recommended Provisions*. Unlike the current design procedure, which is based on U.S. Geological Survey (USGS) peak acceleration and peak velocity-related acceleration ground motion maps developed in the 1970s, the revised design procedure will be based on USGS spectral response maps presently being revised.

The proposed design procedure may take the form of a separate design map based on the new USGS hazard maps or may involve a process specified within the body of the *NEHRP Recommended Provisions* that uses the new USGS maps as a starting point. In developing the design procedure, the BSSC will utilize a process that includes a mechanism to allow for public input, and the draft design procedure will be submitted to the PUC for inclusion in the draft of the 1997 Edition for consensus balloting by the BSSC member organizations. The final design procedure will address design issues such as truncation of high values and establishment of minimum values as well as the issue of using deterministic values.

Essentially, this task will be conducted with the cooperation of the USGS, and the BSSC and USGS have signed a Memorandum of Understanding (MOU) that formalizes the process and specifies that the USGS will hold a series of regional workshops on the current seismic hazard maps and will document the results of those workshops, including comments received and how they were addressed. Based on the results of those workshops, the USGS will revise its seismic hazard maps to reflect the state of the art. The MOU also specifies that the BSSC will include a copy of the revised seismic hazard maps in the 1997 *Provisions* or *Commentary*.

The BSSC, with input from FEMA and USGS, is appointing a 5-member Management Committee (MC) to guide the design procedure effort. The MC membership will be geographically balanced insofar as practicable and will comprise two seismic hazard definition experts and three engineering design experts, including the chairman of the BSSC 1997 PUC. A Resource Group (RG) consisting of interested members from the design, construction, and earth science communities also is being established. Resource Group membership is expected to include representatives of at least the following organizations and agencies: Applied Technology Council, Building Officials and Code Administrators International, Center for Earthquake Research and Information, Earthquake Engineering Research Center, Earthquake Engineering Research Institute, International Conference of Building Officials, National Center for Earthquake Engineering Research, Southern California Earthquake Center, Southern Building Code Congress International, and relevant Structural Engineers Associations (i.e., California, Illinois, Washington).

A 12-member Seismic Design Procedure Group (SDPG) will be responsible for development of the design procedure. The SDPG will be composed of representatives of the different segments of the design community as well as two earth science members designated by the USGS, and the membership will be representative of the different geographical regions of the country.

As part of the process of developing the seismic design procedure, the BSSC will conduct five regional workshops to solicit regional input. A workshop will be held in each of the following regions of the country: Northeast/Southeast, Central States, Wasatch Fault, Pacific Northwest, and California. These workshops will be structured to solicit, examine, and resolve regional issues related to the development of the design procedure and to introduce and begin to obtain consensus on the framework of the design procedure. Information obtained as a result of these workshops will be reflected by the SDPG in the draft design procedure. To facilitate full understanding of the SDPG process and product by the PUC and its TSs, a PUC Executive Committee will be appointed to monitor the SDPG's efforts and report to the full PUC at regular intervals.

Upon completion of the draft design procedure, the BSSC will submit the proposed procedure to the PUC in time for inclusion in the ballot package of the 1997 *Provisions* to be submitted to the BSSC member organizations for balloting.

All technical subcommittee proposals for change are expected to be submitted to the PUC by September 1996 and the PUC will meet twice to consider these proposals and to formulate its recommendations to the BSSC Board of Direction concerning proposals to be submitted to the BSSC member organizations for balloting. The BSSC Board will consider the PUC recommendations at a meeting in late November 1996. The draft ballot package, which serves as a summary of recommended changes, will be reviewed in December 1996. It will be finalized in January 1997 in preparation for the first balloting by the BSSC member organizations. Two rounds of balloting are planned (in February-March 1997 and August 1997).

The balloting by the BSSC member organizations will be conducted according to the BSSC Charter. The results of this ballot will be assembled for review by the PUC and its TSs. These committees will assess the ballot results; resolve, insofar as practicable, any remaining issues for reballoting by BSSC member organizations; and, if necessary, identify technical issues in need of study during subsequent updating of the *NEHRP Recommended Provisions*.

The final consensus version of the 1997 *NEHRP Recommended Provisions* (including as an appendix a report on the differences between the 1994 and 1997 Editions) will be prepared, reviewed, and transmitted to FEMA no later than December 31, 1997.

Information Dissemination

In 1987 a special effort was mounted to stimulate widespread use of the *Provisions*. Particular emphasis was placed on developing the seismic hazard awareness of building owners, developers, insurers, and investors; building and community officials; and key public interest groups.

A series of *Seismic Considerations* handbooks was developed to generate interest in seismic hazard mitigation among the owners and other decision-makers and the design professionals responsible for five building types--apartment buildings, elementary and secondary schools, health care facilities, hotels and motels, and office buildings.

In developing these handbooks, the BSSC involved the national organizations reflecting the interests of the identified groups. These included the Alliance of American Insurers, the American Hospital Association, the American Hotel and Motel Association, the American Institute of Architects, the American Institute for Property and Liability Underwriters, the American Insurance Association's American Insurance Services Group, the American Planning Association, the American School Boards Association, the American Society for Hospital Engineering, the Building Owners and Managers Association, the Council of Educational

Facility Planners International, the Federation of American Health Systems, the Institute of Real Estate Management, the Insurance Information Institute, the International City Management Association, the National Committee on Property Insurance, the National Association of Counties, the National Governors' Association, the National Voluntary Organizations Active in Disasters, The Parent-Teacher Association, and the Public Risk and Insurance Management Association.

These specific efforts were supported by the participation of BSSC representatives in a wide variety of relevant meetings and conferences, BSSC participation in development of curriculum for a FEMA Emergency Management Institute course on the *Provisions* for structural engineers and other design professionals, issuance of a number of press releases, development of in-depth articles for the publications of relevant groups, and the establishment of a computer data base to permit the quick retrieval of various types of information.

In October 1989, the BSSC received from FEMA a request for a proposal to continue its information dissemination effort with emphasis on promoting a seminar series on application of the *NEHRP Recommended Provisions* (based on the Train-the-Trainer Program prepared by FEMA's Emergency Management Institute with the assistance of several BSSC Board members and volunteers) among relevant professional associations, stimulating interest in cosponsorship of the seminars, and conducting the seminars in various locations.

The proposal for initiating this effort was submitted in December 1989, and a contract was received in March 1990. It provided for increasing substantive knowledge about the *NEHRP Recommended Provisions* among a variety of audiences through the organization and conduct of 12 seminars in a variety of locations. In June 1991, in response to a request from FEMA, the BSSC submitted a proposal for continuation of the series with an additional 12 seminars.

By the end of 1993, 32 seminars had been held. Cosponsors included the AIA Building Performance and Regulations Committee, the American Society of Civil Engineers, the American Concrete Institute, the American Institute of Steel Construction, the Building Officials and Code Administrators International (BOCA), the Earthquake Engineering Research Institute Great Lakes Chapter, the Interagency Committee for Seismic Safety in Construction, the Maine Emergency Management Institute, the Masonry Institute of Tennessee, the Materials Handling Institute, the Mississippi State University Continuing Education Department, the Panama Canal Commission, the Portland Cement Association, the Southern Building Code Congress International and Rust International, the Structural Engineers Association of Illinois, and the University of Arkansas Continuing Education Department.

The BSSC's information dissemination effort also provides for conduct of seismic mitigation demonstration projects. The goal of this activity is to enrich the ongoing information dissemination efforts by providing tangible examples of the willingness and ability of various political jurisdictions in targeted geographic areas to consider, adopt, and implement the *NEHRP Recommended Provisions*.

Although it is difficult to determine precisely how effective these various efforts have been, the number of BSSC publications distributed certainly provides at least one measure of the level of interest generated. In this respect, the BSSC can report that more than 65,000 publications have been requested since December 1987, and this number is above and beyond those requests for BSSC documents directed to FEMA.

Further, in 1989, the Building Officials and Code Administrators International (BOCA) appointed an ad hoc committee to review and study the 1988 Edition of the *Provisions* in order to develop a comprehensive and consistent position on code requirements for earthquake loads reflecting technology, design practices, and national codes and standards. In addition to six building officials selected by BOCA, the committee included six individuals representing the BSSC (five of whom were Board members). By October 1990, this group had developed proposed code changes that reflect approximately 90 percent of the content of the *Provisions*. At its annual meeting in September 1991, BOCA adopted new seismic provisions for the *National Building Codes* based on changes proposed by the ad hoc committee. The Southern Building Code Congress International also acted to approve similar seismic provisions for the *Standard Building Code* on October 30, 1991, during its annual meeting. SBCCI's action on the new seismic provisions must be confirmed by a majority of the active members by written ballot. Thus, in essence all three model codes now

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IMPROVING THE SEISMIC SAFETY OF EXISTING BUILDINGS

In August 1991, NIBS entered into a cooperative agreement with FEMA for a comprehensive program leading to the development of a set of nationally applicable guidelines for the seismic rehabilitation of existing buildings. Under this agreement, the BSSC serves as program manager and will cooperate with the American Society of Civil Engineers and the Applied Technology Council in what is expected to be a five-year effort. Initially, FEMA provided funding for a program definition activity designed to generate the detailed work plan for the overall program.

The work plan was completed in April 1992 and in September FEMA contracted with NIBS for the remainder of the effort. The major objectives of the project are to develop a set of technically sound, nationally applicable guidelines (with commentary) for the seismic rehabilitation of buildings; develop building community consensus regarding the guidelines; and develop the basis of a plan for stimulating widespread acceptance and application of the guidelines.

The guidelines document produced as a result of this project is expected to be formulated to serve as a primary resource on the seismic rehabilitation of buildings for the use of model code and standards organizations, state and local building regulatory personnel, design professionals, and educators. The project work, as delineated in the workplan, will, as a minimum, involve ASCE and ATC as subcontractors as well as groups of volunteer experts and paid consultants. The workplan covers all the tasks specified in the cooperative agreement in terms of accomplishment of the three project objectives. The work is structured to ensure that the technical guidelines writing effort will benefit from: consideration of the results of completed and ongoing technical efforts and research activities as well as societal issues, public policy concerns, and the recommendations presented in an earlier FEMA-funded report on issues identification and resolution; cost data on application of rehabilitation procedures; the reactions of potential users; and consensus review by a broad spectrum of building community interests.

To ensure continuing project oversight, a Project Oversight Committee (POC) is responsible to the BSSC Board of Direction for accomplishment of the project objectives and the conduct of project tasks. Further, an advisory panel composed of approximately 20 individuals (plus corresponding members) selected for their knowledge of various aspects of project work (architectural components, systems, cladding; codes and standards; concrete; contractors/constructors; earthquake research; economics; electrical; federal agencies; financing/insurance; historic properties; legal concerns; masonry; mechanical; property owners and managers; seismic hazards; societal concerns and public policy issues; state and local government; steel; structural design/analysis; wood) is being established to review project products and to advise the POC and, if appropriate, the BSSC Board, on the approach being taken, problems arising or anticipated, and progress being made.

While overall management remains the responsibility of the BSSC, responsibility for conduct of the specific project tasks will be shared by the BSSC with ASCE and ATC. Specific BSSC tasks will be completed under the guidance of a BSSC Project Committee.

An earlier FEMA-funded project was designed to provide consensus-backed approval of publications on seismic hazard evaluation and strengthening techniques for existing buildings. This effort involved identifying and resolving major technical issues in two preliminary documents developed for FEMA by others--a handbook for seismic evaluation of existing buildings prepared by the Applied Technology Council (ATC) and a handbook of techniques for rehabilitating existing buildings to resist seismic forces prepared by URS/John A. Blume and Associates (URS/Blume); revising the documents for balloting by the BSSC membership; balloting the documents in accordance with the BSSC Charter; assessing the ballot results; developing proposals to resolve the issues raised; identifying any unresolvable issues; and preparing copies of the documents that reflect the results of the balloting and a summary of changes made and unresolved issues. Basically, this consensus project was directed by the BSSC Board and a 22-member Retrofit of Existing Buildings (REB) Committee composed of individuals representing the needed disciplines and geographical areas and possessing special expertise in the seismic rehabilitation of existing buildings. The consensus approved documents (the *NEHRP Handbook for the Seismic Evaluation of Existing Buildings* and the *NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings*) were transmitted to FEMA in mid-1992.

The BSSC also was involved in the joint venture with the ATC and the Earthquake Engineering Research Institute to develop an action plan for reducing earthquake hazards to existing buildings. It is the action plan that resulted from this effort that prompted FEMA to fund a number of projects, including those described above.

IMPROVING THE SEISMIC SAFETY OF NEW AND EXISTING LIFELINES

Given the fact that buildings continue to be useful in a seismic emergency only if the services on which they depend continue to function, the BSSC developed an action plan for the abatement of seismic hazards to lifelines to provide FEMA and other government agencies and private sector organizations with a basis for their long-range planning. The action plan was developed through a consensus process utilizing the special talents of individuals and organizations involved in the planning, design, construction, operation, and regulation of lifeline facilities and systems. Five lifeline categories were considered: water and sewer facilities, transportation facilities, communication facilities, electric power facilities, and gas and liquid fuel lines. A workshop involving more than 65 participants and the preparation of over 40 issue papers was held. Each lifeline category was addressed by a separate panel and overview groups focused on political, economic, social, legal, regulatory, and seismic risk issues. An Action Plan Committee composed of the chairman of each workshop panel and overview group was appointed to draft the final action plan for review and comment by all workshop participants. The project reports, including the action plan and a definitive six-volume set of workshop proceedings, were transmitted to FEMA in May 1987. In recognition of both the complexity and importance of lifelines and their susceptibility to disruption as a result of earthquakes and other natural hazards (hurricanes, tornadoes, flooding), FEMA subsequently concluded that the lifeline problem could best be approached through a nationally coordinated and structured program aimed at abating the risk to lifelines from earthquakes as well as other natural hazards. Thus, in 1988, FEMA asked the BSSC's parent institution, the National Institute of Buildings Sciences, to provide expert recommendations concerning appropriate and effective strategies and approaches to use in implementing such a program.

The effort, conducted for NIBS by an ad hoc Panel on Lifelines with the assistance of the BSSC, resulted in a report recommending that the federal government, working through FEMA, structure a nationally coordinated, comprehensive program for mitigating the risk to lifelines from seismic and other natural hazards that focuses on awareness and education, vulnerability assessment, design criteria and standards, regulatory policy, and continuing guidance. Identified were a number of specific actions to be taken during the next three to six years to initiate the program. In September 1990, FEMA asked for additional NIBS guidance concerning the feasibility of establishing a national lifelines seismic safety council.

MULTIHAZARD ASSESSMENT

In 1993, FEMA contracted with NIBS for the BSSC to organize and hold a forum intended to explore how best to formulate an integrated approach to mitigating the effects of various natural hazards under the National Earthquake Hazards Reduction Program. Attending the forum, which was held in June 1994, were: Krishna Banga, Department of Veterans Affairs; B. Bienkiewicz, Colorado State University; Gregg Borchelt, Brick Institute of America; John Bryan, Consultant; Arthur Chiu, University of Hawaii; Ronald Cook, University of Florida; Paul Cogswell, Insurance Institute of Property Loss Reduction; Stanley Couvillon, Industrial Risk Insurance; S. K. Ghosh, Portland Cement Association; Charles Goldsmith, Roofing Industry Committee on Wind Issues; Melvyn Green, Melvyn Green and Associates; Michael Hagerty, Bureau of Buildings, City of Portland, Oregon; David A. Harris, NIBS; Brian Hyde, Colorado Water Conservation Board; Gerald H. Jones, Vice Chairman, NIBS Board of Directors; Steve Kelley, Wiss Janney Elstner; Bob Kistner, Colorado Office of Emergency Management; H. S. Lew, National Institute of Standards and Technology; Gabor Lorant, Gabor Lorant Architect; Greg Luth, TSDC; Michael Mahoney, FEMA; Tom McCarty, Factory Mutual Research Corporation; Bob McCluer, Building Officials and Code Administrators International; Kishor Mehta, Institute for Disaster Research, Texas Technical University; Robert Miller, Western Office, National Fire Protection Association; Joseph Minor, University of Missouri, Walter Moore, Walter Moore and Associates; Jim Murphy, Dewberry and Davis; Mary Fran Myers, National Hazards Research and Applications Information Center; Sherry Oaks, Colorado State University; Clifford Oliver, FEMA; William Petak, Institute for Safety Management, University of Southern California; Leon Przbyla, Underwriters Laboratories, Inc.; Lawrence Reaveley, University of Utah; Chistopher Rojahn, Applied Technology Council; Robert Schuster, U.S. Geological Survey; Jim Sealey; Randy Shackleford, Windstorm Inspections, Property Division, Texas Department of Insurance; Douglas Smits, City of Charleston, South Carolina; Jack Snell, National Institute of Standards and Technology; Charles Thornton, The LZA Group Inc.; Richard Vognild, Southern Building Code Congress International; French Wetmore, French and Associates, Ltd.; John Wiggins, J. H. Wiggins Company; Joel Zingeser, Building Technology Inc.

The final report on the forum was delivered to FEMA in early 1995.

BSSC PUBLICATIONS

Available in limited quantity free of charge from the Building Seismic Safety Council, 1201 L Street, N.W., Suite 400, Washington, D.C. 20005

New Buildings

The NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for Seismic Regulations for New Buildings, 1994 Edition, 2 volumes and maps (FEMA Publications 222A and 223B).

The NEHRP (National Earthquake Hazards Reduction Program) Recommended Provisions for the Development of Seismic Regulations for New Buildings, 1991 Edition, 2 volumes and maps (FEMA Publications 222 and 223).

Guide to Application of the 1991 Edition of the NEHRP Recommended Provisions in Earthquake Resistant Building Design, Revised Edition, 1995 (FEMA Publication 140) -- 1995

A Nontechnical Explanation of the NEHRP Recommended Provisions, Revised Edition, 1995 (FEMA Publication 99) -- 1995

Societal Implications:

Seismic Considerations for Communities at Risk, Revised Edition, 1995 (FEMA Publication 83) -- 1995
Seismic Considerations: Apartment Buildings, Revised Edition, 1995 (FEMA Publication 152) -- 1995
Seismic Considerations: Elementary and Secondary Schools, Revised Edition, 1990 (FEMA Publication 149)
Seismic Considerations: Health Care Facilities, Revised Edition, 1990 (FEMA Publication 150)
Seismic Considerations: Hotels and Motels, Revised Edition, 1990 (FEMA Publication 151)
Seismic Considerations: Office Buildings, Revised Edition, 1995 (FEMA Publication 153) -- 1995
Societal Implications: Selected Readings, 1985 (FEMA Publications 84)

Existing Buildings

NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings, 1992 (FEMA Publication 172)

NEHRP Handbook for the Seismic Evaluation of Existing Buildings, 1992 (FEMA Publication 178)

An Action Plan for Reducing Earthquake Hazards of Existing Buildings, 1985 (FEMA Publication 90)

Lifelines

Abatement of Seismic Hazards to Lifelines: An Action Plan, 1987 (FEMA Publication 142)

Abatement of Seismic Hazards to Lifelines: Proceedings of a Workshop on Development of An Action Plan, 6 volumes:

Papers on Water and Sewer Lifelines, 1987 (FEMA Publication 135)
Papers on Transportation Lifelines, 1987 (FEMA Publication 136)
Papers on Communication Lifelines, 1987 (FEMA Publication 137)
Papers on Power Lifelines, 1987 (FEMA Publication 138)
Papers on Gas and Liquid Fuel Lifelines, 1987 (FEMA Publication 139)
Papers on Political, Economic, Social, Legal, and Regulatory Issues and General Workshop Presentations, 1987 (FEMA Publication 143)

BSSC MEMBER ORGANIZATIONS

AFL-CIO Building and Construction Trades Department
AISC Marketing, Inc.
American Concrete Institute
American Consulting Engineers Council
American Forest and Paper Association
American Institute of Architects
American Institute of Steel Construction
American Insurance Services Group, Inc.
American Iron and Steel Institute
American Plywood Association
American Society of Civil Engineers
Applied Technology Council
Associated General Contractors of America
Association of Engineering Geologists
Association of Major City Building Officials
Bay Area Structural, Inc.*
Brick Institute of America
Building Officials and Code Administrators International
Building Owners and Managers Association International
Building Technology, Incorporated*
California Geotechnical Engineers Association
Canadian National Committee on Earthquake Engineering
Concrete Masonry Association of California and Nevada
Concrete Reinforcing Steel Institute
Earthquake Engineering Research Institute
General Reinsurance Corporation*
Interagency Committee on Seismic Safety in Construction
International Conference of Building Officials
Masonry Institute of America
Metal Building Manufacturers Association
National Association of Home Builders
National Concrete Masonry Association
National Conference of States on Building Codes and Standards
National Elevator Industry, Inc.
National Fire Sprinkler Association
National Institute of Building Sciences
National Ready Mixed Concrete Association
Permanent Commission for Structural Safety of Buildings*
Portland Cement Association
Precast/Prestressed Concrete Institute
Rack Manufacturers Institute
Seismic Safety Commission (California)
Southern Building Code Congress International
Southern California Gas Company*
Steel Deck Institute, Inc.
Steel Joist Institute*
Steven Winter Associates, Inc.*
Structural Engineers Association of Arizona
Structural Engineers Association of California
Structural Engineers Association of Central California
Structural Engineers Association of Colorado
Structural Engineers Association of Illinois
Structural Engineers Association of Northern California
Structural Engineers Association of Oregon
Structural Engineers Association of San Diego
Structural Engineers Association of Southern California
Structural Engineers Association of Utah
Structural Engineers Association of Washington
The Masonry Society
U. S. Postal Service*
Western States Clay Products Association
Western States Council Structural Engineers Association
Westinghouse Electric Corporation*

* Affiliate (non-voting) members.

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