NEHRP Recommended Provisions

for Seismic Regulations for New Buildings and Other Structures

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Part 2: Commentary

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Chapter 1 Commentary

GENERAL PROVISIONS

Chapter 1 sets forth general requirements for applying the analysis and design provisions contained in Chapters 2 through 14 of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*. 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, 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 *Provisions* is intended to serve as a resource 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 Use 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 *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 an unspecified "authority having jurisdiction." 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. 1.1.2.5 where the need for well-established criteria and systems of testing and approval are recognized even though few such systems are in place. In some instances, the decision-making mechanism referred to is clearly most logically the province of a building official or department; in others, it 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 "authority having jurisdiction" 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 Appendix A to this *Commentary* volume presents a detailed explanation of the development of *Provisions* Maps 1 through 24 and Appendix B describes development of the U.S. Geological Survey seismic hazard maps on which the *Provisions* maps are based. An overview of the Building Seismic Safety Council (BSSC) and its activities appears at the end of the volume.

1.1 GENERAL

1.1.1 Purpose. The goal of the *Provisions* is to present criteria for the design and construction of new structures subject to earthquake ground motions in order to minimize the hazard to life for all structures, to increase the expected performance of structures having a substantial public hazard due to occupancy or use 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 for the protection of life safety in structures subject to earthquakes. The *Provisions* document has been reviewed extensively and balloted by the architectural, engineering, and construction communities and, therefore, it is a proper source for the development of building codes in areas of seismic exposure.

Some design standards go further than the *Provisions* and attempt to minimize damage as well as protect building occupants. For example, the *California Building Code* has added property protection in relation to the design and construction of hospitals and public schools. The *Provisions* document generally considers property damage as it relates to occupant safety for ordinary structures. 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 in the structural system. For ground motions in excess of the design levels, the intent of the *Provisions* is for the structure to have 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 for most structures. However, a high degree of life safety, albeit with some structural and nonstructural damage, can be achieved economically in structures by allowing inelastic energy dissipation in the structure. The objective of the *Provisions* therefore is to set forth the minimum requirements to provide reasonable and prudent life safety. 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, but the damage may not be economically repairable.

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.

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

1. Detached one- and two-family dwellings in Seismic Design Categories A, B, and C are exempt because they represent low seismic risks.
2. Structures constructed using the conventional light-frame construction requirements in Sec. 12.5 are deemed capable of resisting the seismic forces imposed by the *Provisions.* While specific elements of conventional light-frame construction may be calculated to be overstressed, there is typically a great deal of redundancy and uncounted resistance in such structures. Detached one- and two-story wood-frame dwellings have generally performed well even in regions of higher seismicity. The requirements of Sec. 12.5 are adequate to provide the safety required for such dwellings without imposing any additional requirements of the *Provisions*.
3. Agricultural storage structures are generally exempt from most code requirements because of the exceptionally low risk to life involved and that is the case of the *Provisions*.
4. Structures in areas with extremely low seismic risk need only comply with the design and detailing requirements for structures assigned to Seismic Design Category A.

The *Provisions* are not retroactive and apply only to existing structures when there is an addition, change of use, or alteration. As a minimum, existing structures should comply with legally adopted regulations for repair and rehabilitation as related to earthquake resistance. (Note: Publications such as the *Handbook for the Seismic Evaluation of Buildings—A Prestandard* [FEMA 310] and the *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* [FEMA 356] are available.)

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.

1.1.2.2 Additions. Additions that are structurally independent of an existing structure are considered to be new structures required to comply with the *Provisions*. For additions that are not structurally independent, the intent is that the addition as well as the existing structure 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 structure without bringing the entire structure into compliance with the *Provisions*. Additions also shall not reduce the lateral force resistance of any existing element to less than that required for a new structure.

1.1.2.3 Change of use. When a change in the use of a structure will result in the structure being reclassified to a higher Seismic Use Group, the existing structure must be brought into compliance with the requirements of the *Provisions* as if it were a new structure. Structures in higher Seismic Use Groups are intended to provide a higher level of safety to occupants and in the case of Seismic Use Group III to be capable of performing their safety-related function after a seismic event. An exception is allowed when the change is from Seismic Use Group I to Seismic Use Group II where S_{DS} is less than 0.3. The expense that may be necessary to upgrade such a structure because of a change in the Seismic Use Group cannot be justified for structures located in regions with low seismic risk.

1.1.2.4 Alterations. Alterations include all significant modifications to existing structures that are not classified as an addition. No reduction in strength of the seismic-force-resisting system or stiffness of the structure shall result from an alteration unless the altered structure is determined to be in compliance with the *Provisions*.

Like additions, an increase of not greater than 5 percent of the mass contributing to seismic forces is permitted in any structural element of the existing structure without bringing the entire structure into compliance with the *Provisions*.

The cumulative effects of alterations and additions should not increase the seismic forces in any structural element of the existing structure by more than 5 percent unless the capacity of the element subject to the increased seismic forces is still in compliance with the *Provisions*.

1.1.2.5 Alternate materials and alternate means 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 and historical seismic performance. 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, authorities having jurisdiction 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 structure. These conditions, where applicable, should include several cycles of full reversals of loads and deformations in the inelastic range.

1.2 SEISMIC USE GROUPS

The expected performance of structures shall be controlled by assignment of each structure to one of three Seismic Use Groups. Seismic Use Groups are categorized based on the occupancy of the structures within the group and the relative consequences of earthquake-induced damage to the structures. The *Provisions* specify progressively more conservative strength, drift control, system selection, and detailing requirements for structures contained in the three groups, in order to attain minimum levels of earthquake performance suitable to the individual occupancies.

In previous editions of the *Provisions*, this categorization of structures, by occupancy, or use, was termed a Seismic Hazard Exposure Group. The name Seismic Use Group was adopted in the 1997 *Provisions* as being more representative of the definition of this classification. Seismic hazard relates to the severity and frequency of ground motion expected to affect a structure. Since structures contained in these groups are spread across the various zones of seismicity, from high to low hazard, the groups do not really relate to hazard. Rather the groups, categorized by occupancy or use, are used to establish design criteria intended to produce specific types of performance in design earthquake events, based on the importance of reducing structural damage and improving life safety.

In terms of post-earthquake recovery and redevelopment, certain types of occupancies are vital to public needs. These special occupancies were identified and given specific recognition. In terms of disaster preparedness, regional communication centers identified as critical emergency services should be in a higher classification than retail stores, office buildings, and factories.

Specific consideration is given to Group III, essential facilities required for post-earthquake recovery. Also included are structures that contain substances, that if released into the environment, are deemed to be hazardous to the public. The 1991 Edition included a flag to urge consideration of the need for utility services after an earthquake. It is at the discretion of the authority having jurisdiction which structures are required for post-earthquake response and recovery. This is emphasized with the term “designated” before many of the structures listed in Sec. 1.2.1. Using Item 3, “designated medical facilities having emergency treatment facilities,” as an example, the authority having jurisdiction should inventory medical facilities having emergency treatment facilities within the jurisdiction and designate those to be required for post-earthquake response and recovery. In a rural location where there may not be a major hospital, the authority having jurisdiction may choose to require outpatient surgery clinics to be designated Group III structures. On the other hand, these same clinics in a major jurisdiction with hospitals nearby may not need to be designated Group III structures.

Group II structures are those having a large number of occupants and those where the occupants' ability to exit is restrained. The potential density of public assembly uses in terms of number of people warrant an extra level of care. The level of protection warranted for schools, day care centers, and medical facilities is greater than the level of protection warranted for occupancies where individuals are relatively self-sufficient in responding to an emergency.

Group I contains all uses other than those excepted generally from the requirements in Sec. 1.1.2.1. Those in Group I have lesser life hazard only insofar as there is the probability of fewer occupants in the structures and the structures are lower and/or smaller.

In structures with multiple uses, the 1988 Edition of the *Provisions* required that the structure be assigned the classification of the highest group occupying 15 percent or more of the total area of the structure. This was changed in the 1991 Edition to require the structure to be assigned to the highest group present. These requirements were further modified to allow different portions of a structure to be assigned different Seismic Use Groups provided the higher group is not negatively impacted by the lower group. When a lower group impacts a higher group, the higher group must either be seismically independent of the other, or the two must be in one structure designed seismically to the standards of the higher group. Care must be taken, however, for the case in which the two uses are seismically independent but are functionally dependent. The fire and life-safety requirements relating to exiting, occupancy, fire-resistive construction and the like of the higher group must not be reduced by interconnection to the lower group. Conversely, one must also be aware that there are instances, although uncommon, where certain fire and life-safety requirements for a lower group may be more restrictive than those for the higher group. Such assignments also must be considered when changes are made in the use of a structure even though existing structures are not generally within the scope of the *Provisions*.

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 performance desired of structures in these individual groups.

Although the *Provisions* explicitly require design for only a single level of ground motion, it is expected that structures designed and constructed in accordance with these requirements will generally be able to meet a number of performance criteria, when subjected to earthquake ground motions of differing severity. The performance criteria discussed here were jointly developed during the BSSC Guidelines and Commentary for Seismic Rehabilitation of Buildings Project (ATC, 1995) and the Structural Engineers Association of California Vision 2000 Project (SEAOC, 1995). In the system established by these projects, earthquake performance of structures is defined in terms of several standardized performance levels and reference ground motion levels. Each performance level is defined by a limiting state in which specified levels of degradation and damage have occurred to the structural and nonstructural building components. The ground motion levels are defined in terms of their probability of exceedance.

Although other terminology has been used in some documents, four performance levels are commonly described as meaningful for the design of structures. These may respectively be termed the operational, immediate occupancy, life safety, and collapse prevention levels. Of these, the operational level represents the least level of damage to the structure. Structures meeting this level when responding to an earthquake are expected to experience only negligible damage to their structural systems and minor damage to nonstructural systems. The structure will retain nearly all of its pre-earthquake strength and stiffness and all mechanical, electrical, plumbing, and other systems necessary for the normal operation of the structure are expected to be functional. If repairs are required, these can be conducted at the convenience of the occupants.

The risk to life safety during an earthquake in a structure meeting this performance level is negligible. Note, that in order for a structure to meet this level, all utilities required for normal operation must be available, either through standard public service or emergency sources maintained for that purpose. Except for very low levels of ground motion, it is generally not practical to design structures to meet this performance level.

The immediate occupancy level is similar to the operational level although somewhat more damage to nonstructural systems is anticipated. Damage to the structural systems is very slight and the structure retains all of its pre-earthquake strength and nearly all of its stiffness. Nonstructural elements, including ceilings, cladding, and mechanical and electrical components, remain secured and do not represent hazards. Exterior nonstructural wall elements and roof elements continue to provide a weather barrier, and to be otherwise serviceable. The structure remains safe to occupy; however, some repair and clean-up is probably required before the structure can be restored to normal service. In particular, it is expected that utilities necessary for normal function of all systems will not be available, although those necessary for life safety systems would be provided. Some equipment and systems used in normal function of the structure may experience internal damage due to shaking of the structure, but most would be expected to operate if the necessary utility service was available. Similar to the operational level, the risk to life safety during an earthquake in a structure meeting this performance level is negligible. Structural repair may be completed at the occupants' convenience, however, significant nonstructural repair and cleanup is probably required before normal function of the structure can be restored.

At the life safety level, significant structural and nonstructural damage has occurred. The structure may have lost a substantial amount of its original lateral stiffness and strength but still retains a significant margin against collapse. The structure may have permanent lateral offset and some elements of the seismic-force-resisting system may exhibit substantial cracking, spalling, yielding, and buckling. Nonstructural elements of the structure, while secured and not presenting falling hazards, are severely damaged and cannot function. The structure is not safe for continued occupancy until repairs are instituted as strong ground motion from aftershocks could result in life threatening damage. Repair of the structure is expected to be feasible, however, it may not be economically attractive to do so. The risk to life during an earthquake, in a structure meeting this performance level is very low.

At the collapse prevention level a structure has sustained nearly complete damage. The seismic-force-resisting system has lost most of its original stiffness and strength and little margin remains against collapse. Substantial degradation of the structural elements has occurred including extensive cracking and spalling of masonry and concrete elements and buckling and fracture of steel elements. The structure may have significant permanent lateral offset. Nonstructural elements of the structure have experienced substantial damage and may have become dislodged creating falling hazards. The structure is unsafe for occupancy as even relatively moderate ground motion from aftershocks could induce collapse. Repair of the structure and restoration to service is probably not practically achievable.

The design ground motion contained in the *Provisions* is taken as two-thirds of the maximum considered earthquake ground motion. Such ground motion may have a return period varying from a few hundred years to a few thousand years, depending on the regional seismicity. It is expected that structures designed in accordance with the requirements for Group I would achieve the life safety or better performance level for these ground motions. Structures designed in accordance with the requirements for Group III should be able to achieve the Immediate Occupancy or better performance level for this ground motion. Structures designed to the requirements for Group II would be expected to achieve performance better than the life safety level but perhaps less than the immediate occupancy level for this ground motion.

While the design ground motion represents a rare earthquake event, it may not be the most severe event that could ever affect a site. In zones of moderate seismicity, it has been common practice in the past to consider ground motion with a 98 percent chance of non-exceedance in 50 years, or an average return period of 2,500 years, as being reasonably representative of the most severe ground motion ever likely to affect a site. This earthquake has been variously termed a maximum credible earthquake, maximum capable event and, most recently, a maximum considered earthquake. The recent terminology is adopted here in recognition that ground motion of this probability level is not the most severe motion that could ever effect the site, but is considered sufficiently improbable that more severe ground motions need not practically be considered. In regions near major active faults, such as coastal California, estimates of ground motion at this probability of exceedance can produce structural demands much larger than has typically been recorded in past earthquakes. Consequently, in these zones, the maximum considered earthquake is now commonly taken based on conservative estimates of the ground motion from a deterministic event, representing the largest magnitude event that the nearby faults are believed capable of producing.

It is expected that structures designed to the requirements for Group I would be capable of responding to the maximum considered earthquake at a near collapse or better performance level. Structures designed to the requirements for Group III should be capable of responding to such ground motions at the life safety level. Structures designed and constructed to the requirements for Group II structures should be capable of responding to maximum considered earthquake ground motions with a performance intermediate to the near collapse and life safety levels.

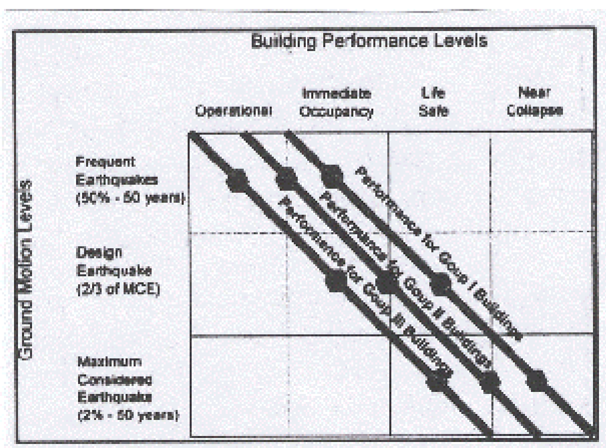

Figure 1.2-1 Expected building performance.

In zones of high seismicity, structures may experience strong motion earthquakes several times during their lives. It is also important to consider the performance expected of structures for these somewhat less severe, but much more frequent, events. For this purpose, earthquake ground shaking with a 50 percent probability of non-exceedance in 50 years may be considered. Sometimes termed a maximum probable event (MPE), such ground motion would be expected to recur at a site, one time, every 72 years. Structures designed to the requirements for Group I would be expected to respond to such ground motion at the Immediate Occupancy level. Structures designed and constructed to either the Group II or Group III requirements would be expected to perform to the Operational level for these events. This performance is summarized in Figure C1.2-1.

It is important to note that while the performance indicated in Figure C1.2-1 is generally indicative of that expected for structures designed in accordance with the *Provisions,* there can be significant variation in the performance of individual structures from these expectations. This variation results from individual site conditions, quality of construction, structural systems, detailing, overall configuration of the structure, inaccuracies in our analytical techniques, and a number of other complex factors. As a result of these many factors, and intentional conservatism contained in the *Provisions*, most structures will perform better than indicated in the figure and others will not perform as well.

1.2.5 Seismic Use Group III structure access protection. This section establishes the requirement for access protection for Seismic Use Group III structures. There is a need for ingress/egress to those structures that are essential post-earthquake facilities and this shall be considered in the siting and design of the structure.

1.3 OCCUPANCY IMPORTANCE FACTOR

Although the concept of an occupancy importance factor for structural systems has been included in the *Uniform Building Code* for many years, it was first adopted into the 1997 Edition of the *Provisions.* The inclusion of the occupancy importance factor is one of several requirements included in this edition of the *Provisions* where there are attempts to control the seismic performance capability of structures in the different Seismic Use Groups. Specifically, the occupancy importance factor modifies the R coefficients used to determine minimum design base shears. Structures assigned occupancy importance factors greater than 1.0 must be designed for larger seismic forces. As a result, these structures are expected to experience lower ductility demands than structures designed with lower occupancy importance factors and, thus sustain less damage. The *Provisions* also include requirements that attempt to limit vulnerability to structural damage by specifying more stringent drift limits for structures in Seismic Use Groups of higher risk. Further discussion of these concepts is found in *Commentary* Sec. 4.2.1 and 4.5.

1.4 SEISMIC DESIGN CATEGORY

This section establishes the design categories that are the keys for establishing design requirements for any structure based on its use (Seismic Use Group) and on the level of expected seismic ground motion. Once the Seismic Design Category (A, B, C, D, E, or F) for the structure is established, many other requirements such as detailing, quality assurance, system limits, height limitations, specialized requirements, and change of use are related to it.

Prior to the 1997 edition of the *Provisions*, these categories were termed Seismic Performance Categories. While the desired performance of the structure, under the design earthquake, was one consideration used to determine which category a structure should be assigned to, it was not the only factor. The seismic hazard at the site was actually the principle parameter that affected a structure’s category. The name was changed to Seismic Design Category to represent the uses of these categories, which is to determine the specific design requirements.

The earlier editions of the *Provisions* utilized the peak velocity-related acceleration, A_v, to determine a building’s Seismic Performance Category. However, this coefficient does not adequately represent the damage potential of earthquakes on sites with soil conditions other than rock. Consequently, the 1997 *Provisions* adopted the use of response spectral acceleration parameters S_{DS} and S_{D1}, which include site soil effects for this purpose. Instead of a single table, as was present in previous editions of the *Provisions*, two tables are now provided, relating respectively to short-period and long-period ground motions.

Seismic Design Category A represents structures in regions where anticipated ground motions are minor, even for very long return periods. For such structures, the *Provisions* require only that a complete seismic-force-resisting system be provided and that all elements of the structure be tied together. A nominal design force equal to 1 percent of the weight of the structure is used to proportion the lateral system.

It is not considered necessary to specify seismic-resistant design on the basis of a maximum considered earthquake ground motion for Seismic Design Category A structures because the ground motion computed for the areas where these structures are located is determined more by the rarity of the event with respect to the chosen level of probability than by the level of motion that would occur if a small but close earthquake actually did occur. However, it is desirable to provide some protection against earthquakes and many other types of unanticipated loadings. Thus, the requirements for Seismic Design Category A provide a nominal amount of structural integrity that will improve the performance of buildings in the event of a possible but rare earthquake even though it is possible that the ground motions could be large enough to cause serious damage or even collapse. The result of design to Seismic Design Category A requirements is that fewer building would collapse in the vicinity of such an earthquake.

The integrity is provided by a combination of requirements. First, a complete load path for lateral forces must be identified. Then it must be designed for a lateral force based on a 1 percent acceleration of the mass. The minimum connection forces specified for Seismic Design Category A also must be satisfied.

The 1 percent value has been used in other countries as a minimum value for structural integrity. For many structures, design for the wind loadings specified in the local buildings codes normally will control the lateral force design when compared to the minimum integrity force on the structure. However, many low-rise, heavy structures or structures with significant dead loads resulting from heavy equipment may be controlled by the nominal 1 percent acceleration. Also, minimum connection forces may exceed structural forces due to wind in some structures.

Seismic Design Category B includes Seismic Use Group I and II structures is regions of seismicity where only moderately destructive ground shaking is anticipated. In addition to the requirements for Seismic Design Category A, structures in Seismic Design Category B must be designed for forces determined using Maps 1 through 24.

Seismic Design Category C includes Seismic Use Group III structures in regions where moderately destructive ground shaking may occur as well as Seismic Use Group I and II structures in regions with somewhat more severe ground shaking potential. In Seismic Design Category C, the use of some structural systems is limited and some nonstructural components must be specifically designed for seismic resistance.

Seismic Design Category D includes structures of Seismic Use Group I, II, and III located in regions expected to experience destructive ground shaking but not located very near major active faults. In Seismic Design Category D, severe limits are placed on the use of some structural systems and irregular structures must be subjected to dynamic analysis techniques as part of the design process.

Seismic Design Category E includes Seismic Use Group I and II structures in regions located very close to major active faults and Seismic Design Category F includes Seismic Use Group III structures in these locations. Very severe limitations on systems, irregularities, and design methods are specified for Seismic Design Categories E and F. For the purpose of determining if a structure is located in a region that is very close to a major active fault, the *Provisions* use a trigger of a mapped maximum considered earthquake spectral response acceleration parameter at 1-second period, S_1, of 0.75 or more regardless of the structure's fundamental period. The mapped short period acceleration, S_S, was not used for this purpose because short period response accelerations do not tend to be affected by near-source conditions as strongly as do response accelerations at longer periods.

Local or regional jurisdictions enforcing building regulations need to consider the effect of the maps, typical soil conditions, and Seismic Design Categories on the practices in their jurisdictional areas. For reasons of uniformity of practice or reduction of potential errors, adopting ordinances could stipulate particular values of ground motion, particular Site Classes, or particular Seismic Design Categories for all or part of the area of their jurisdiction. For example:

1. An area with an historical practice of high seismic zone detailing might mandate a minimum Seismic Design Category of D regardless of ground motion or Site Class.
2. A jurisdiction with low variation in ground motion across the area might stipulate particular values of the ground motion rather than requiring use of the maps.
3. An area with unusual soils might require use of a particular Site Class unless a geotechnical investigation proves a better Site Class.

There are two limits on period for permission to ignore S_{D1} when establishing the Seismic Design Category. The first rule, requiring T_a be less than 80% of T_s, allows some conservatism for the uncertainty in estimating periods. The second rule only applies where a different period is used for computing drift than for computing forces. In that case, the period used for establishing drift must be less than the corner period, T_s. It should be noted that the period used for establishing drift could simply be T_a and, as such, does not require that the actual building period be calculated.

1.4.2 Site limitation for Seismic Design Categories E and F. The forces that result on a structure located astride the trace of a fault rupture that propagates to the surface are extremely large and it is not possible to reliably design a structure to resist such forces. Consequently, the requirements of this section limit the construction of buildings in Seismic Design Categories E and F on sites subject to this hazard. Similarly, the effects of landsliding, liquefaction, and lateral spreading can be highly damaging to a building. However, the effects of these site phenomena can more readily be mitigated through the incorporation of appropriate design measures than can direct ground fault rupture. Consequently, construction on sites with these hazards is permitted if appropriate mitigation measures are included in the design.

1.5 REQUIREMENTS FOR SEISMIC DESIGN CATEGORY A

Because of the very low seismicity associated with sites with S_{DS} less than 0.25 and S_{D1} less than 0.10, it is considered appropriate for Category A buildings to require only a complete seismic-force-resisting system, 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 requirements as being suitable to reduce the windstorm risk. Since the *Provisions* considers only earthquakes, no other requirements are prescribed for Category A buildings. Only a complete seismic-force-resisting system, ties, and wall anchorage are required by these *Provisions*.

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.

1.5.1 Lateral forces. This analysis procedure, which was added to the *Provisions* in the 1997 edition, is applicable only to structures in Seismic Design Category A. Such structures are not designed for resistance to any specific level of earthquake ground shaking as the probability that they would ever experience shaking of sufficient intensity to cause life threatening damage is very low so long as the structures are designed with basic levels of structural integrity. Minimum levels of structural integrity are achieved in a structure by assuring that all elements in the structure are tied together so that the structure can respond to shaking demands in an integral manner and also by providing the structure with a complete seismic-force-resisting system. It is believed that structures having this level of integrity would be able to resist, without collapse, the very infrequent earthquake ground shaking that could affect them. In addition, requirements to provide such integrity provides collateral benefit with regard to the ability of the structure to survive other hazards such as high wind storms, tornadoes, and hurricanes.

The procedure outlined in this section is intended to be a simple approach to ensuring both that a building has a complete seismic-force-resisting system and that it is capable of sustaining at least a minimum level of lateral force. In this analysis procedure, a series of static lateral forces equal to 1 percent of the weight at each level of the structure is applied to the structure independently in each of two orthogonal directions. The structural elements of the seismic-force-resisting system then are designed to resist the resulting forces in combination with other loads under the load combinations specified by the building code.

The selection of 1 percent of the building weight as the design force for Seismic Design Category A structures is somewhat arbitrary. This level of design lateral force was chosen as being consistent with prudent requirements for lateral bracing of structures to prevent inadvertent buckling under gravity loads and also was believed to be sufficiently small as to not present an undue burden on the design of structures in zones of very low seismic activity.

The seismic weight 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 authority having jurisdiction 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 authority having jurisdiction.

1.5.2 Connections. The requirements in this section are a simplified version of the material found in Sec. 4.6.1.1. For Seismic Design Category A, 5 percent is always greater than 0.133 times S_{DS}.

1.5.3 Anchorage of concrete or masonry walls. The intent of this section is to ensure that out-of-plane inertia forces generated within a concrete or masonry wall can be transferred to the adjacent roof or floor construction. The transfer can be accomplished only by reinforcement or anchors.

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Chapter 2 Commentary

QUALITY ASSURANCE

2.1 GENERAL

2.1.1 Scope. Quality assurance (control and verification) for structures assigned to Seismic Design Categories C, D, E and F, is necessary due to the complexity of the seismic-force-resisting systems and is important because of the serious consequences of the failure of structures. The level of quality assurance varies with the degree of seismic risk.

Quality Assurance requirements involve many aspects of the structural design and construction process—from the selection of the design team and their suitability for the project to the capabilities of the construction contractor(s) and subcontractors, whether selected by qualification or by low bid. Where structures are to be located in areas with a high probability of having damaging earthquake ground motion, adequate quality assurance is required to provide life safety. Unfortunately, in recent seismic events there have been numerous earthquake-related failures that are directly traceable to poor design or poor quality control during construction; these deficiencies must be eliminated. The earthquake requirements included in the *Provisions* rely heavily upon the concept of adequate quality control and verification to assure sound construction. It is important that all parties involved in the design and construction process understand and support the quality assurance requirements recommended in the *Provisions*.

The technological complexity of the design of modern structures necessitates employment of a team of registered design professionals. Each member in responsible charge of design of each element or system of the structure must be qualified and licensed by the jurisdiction to practice in their technical fields of practice. Structures located at a site with a potential to have damaging earthquake ground motion must be designed to withstand the resulting seismic forces and accommodate element displacements.

Every element of a structure is a part of a continuous load path transmitting seismic forces from and to the foundations, which must be adequately strengthened and appropriately anchored to resist the seismic forces and to accommodate the resulting displacements. Many of the failures in recent earthquakes have been attributed to weak links in the seismic-force-resisting load paths. Since the connections between adjacent elements of the structure often involve different registered design professionals and different construction trades during installation, it is imperative that these connections be adequately described in the construction documents and observed during installation. In order to accommodate these constraints and produce a coordinated design, the registered design professionals must function as an integrated and well coordinated team.

The selection of the size and configuration of the structure, and the type of structural seismic-force-resisting system(s) selected, can have a significant impact on the performance of the structure in an earthquake. Since the selection can affect the design and cost of construction of almost every element of the structure, it is essential that the entire design team participate in making these preliminary design decisions and appropriately accommodate them in their design. While not required by the *Provisions,* it is recommended that a quality assurance plan be prepared for the design process.

For quality assurance during construction, the following is included in the *Provisions*: (1) the registered design professional(s) in responsible charge of the design specifies the quality assurance requirements; (2) the prime contractor(s) exercises the control necessary to achieve the required quality; and (3) the owner monitors the construction process by means of consultants who perform special inspections, observations, and testing. It is important that all of the parties involved recognize their responsibilities, understand the procedures, and are capable of carrying them out. Because the contractor and specialty subcontractors are performing the work and exercising control of quality, it is essential that the special inspections and tests be performed by someone not in their direct employ. For this reason, the special inspectors are the owner's inspectors and serve at the discretion of the authority having jurisdiction. When the owner is also the contractor, the owner, to avoid a potential conflict of interest, must engage independent agencies to conduct the special inspections and tests rather than try to qualify his own employees for that purpose.

The contractual responsibilities during the construction phase vary from project to project depending on the structure, and the desires of the owner. The majority of building owners use the standard contract forms published by the American Institute of Architects (AIA) or the Engineers' Joint Contract Documents Committee (EJCDC) (or a contract modeled therefrom) which include specific construction phase responsibilities.

The registered design professional in responsible charge for each portion of the project is the most knowledgeable person available for assuring appropriate conformance with the intent of the design as conveyed in the construction documents. It is essential that a registered design professional be sufficiently involved during the construction phase of the project to assure general conformance with the approved construction documents. Courts are ruling more frequently that the above responsibilities remain that of the registered design professional in responsible charge of the design regardless of the language included in the contract for professional services.

The quality assurance requirements included in Chapter 2 of the *Provisions* are the minimum requirements. It could be the decision of the owner or registered design professional to include more stringent quality assurance requirements. The primary method for achieving quality assurance is through the use of special inspectors and testing agencies.

Registered design professional(s) in responsible charge, or their employees, may perform the special inspections, when approved by the authority having jurisdiction. Increased involvement by the registered design professional in responsible charge allows for early detection of problems during construction when they can be resolved more easily.

2.2 GENERAL REQUIREMENTS

Because of the complexity of design and construction for structures included in Seismic Design Categories C, D, E, and F, it is necessary to provide a comprehensive written quality assurance plan to assure adequate quality controls and verification during construction. Each portion of the quality assurance plan is required to be prepared by the registered design professional responsible for the design of the seismic-force-resisting system(s) and other designated seismic systems that are subject to requirements for quality assurance. When completed, the quality assurance plan must be submitted to the owner and to the authority having jurisdiction.

The performance for quality control of the contractors and subcontractors varies from project to project. The quality assurance plan provides an opportunity for the registered design professional to delineate the types and frequency of testing and inspections, and the extent of the structural observations to be performed during the construction process and to assure that the construction is in conformance with the approved construction documents. Special attention should be given in the quality assurance plan for projects with higher occupancy importance factors.

The authority having jurisdiction shall approve the quality assurance plan and shall obtain from each contractor a written statement that the contractor understands the requirements of the quality assurance plan and will exercise the necessary control to obtain conformance. The exact methods of control are the responsibility of the individual contractors, subject to approval by the authority having jurisdiction. Special inspections, in addition to those included in the quality assurance plan, may be required by the authority having jurisdiction to ensure that there is compliance with the approved construction documents.

As indicated in Sec. 2.2, certain regular, low-rise structures assigned to Seismic Use Group I are exempt from preparation of a quality assurance plan. Any structure that does not satisfy all of the criteria included in the exception or is not otherwise exempted by the *Provisions* is required to have a quality assurance plan. It is important to emphasize that this exemption only applies to the preparation of a quality assurance plan. All special inspections and testing that are otherwise required by the *Provisions* must be performed.

2.3 SPECIAL INSPECTION

Special inspection is the monitoring of materials and workmanship that are critical to the integrity of the structure. The requirements listed in this section, from foundation systems through cold-formed steel framing, have been included in the national model codes for many years. It is a premise of the *Provisions* that there will be an adequate supply of knowledgeable and experienced inspectors available to provide the necessary special inspections 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 authority having jurisdiction as being qualified to perform special inspections for the category of work involved. As a guide to the authority having jurisdiction, it is suggested that the special inspector is to be one of the following:

1. A person employed and supervised by the registered design professional in responsible charge for the design of the designated seismic system or the seismic-force-resisting system for which the special inspector is engaged.
2. A person employed by an approved inspection and/or testing agency who is under the direct supervision of a registered design professional also employed by the same agency, using inspectors or technicians qualified by recognized industry organizations as approved by the authority having jurisdiction.
3. A manufacturer or fabricator of components, equipment, or machinery that has been approved for manufacturing components that satisfy seismic safety standards and that maintain a quality assurance plan approved by authority having jurisdiction. The manufacturer or fabricator is required to provide evidence of such approval by means of clear marks on each designated seismic system or seismic-force-resisting system component shipped to the construction site.

The extent and duration of special inspections, types of testing, and the frequency of the testing must be clearly delineated in the quality assurance plan. In some instances the *Provisions* allow periodic special inspection rather than continuous special inspection. Where periodic special inspections are allowed, the *Provisions* do not state specific requirements for frequency of periodic inspection, but do indicate stages of construction at which inspection is required for a particular category of work. The quality assurance plan should generally indicate the timing and extent of any periodic special inspections required by the *Provisions*.

2.3.9 Architectural components. It is anticipated that the minimum requirements for architectural components (such as exterior cladding) are satisfied if the method of anchoring components and the number, spacing, and types of fasteners used conform to approved construction documents.

For ceilings and access floors compliance with the construction documents should concentrate on critical details. For ceiling grids those details are the location and installation for grid bracing, the connection of runners to the perimeter edge member along two adjacent sides, and the gap provided between ends of runners and the edge member on the remaining two sides.

2.3.10 Mechanical and electrical components. It is anticipated that the minimum requirements for mechanical and electrical components are satisfied if the method of anchoring components and the number, spacing, and types of fasteners actually used conform to the approved construction documents. It is noted that such special inspection requirements are for selected electrical, lighting, piping, and ductwork components in any Seismic Design Category except A or B, and for all electrical equipment in Seismic Design Category E or F.

2.4 TESTING

Compliance with nationally recognized test standards provides the authority having jurisdiction and the owner a means to determine the acceptability of materials and their placement. Most test standards for materials are developed and maintained by the American Society for Testing and Materials (ASTM). Through their reference in model building codes and material specifications, ASTM Standards and other standard testing procedures provide a uniform measure for acceptance of materials and construction. The *Provisions* and the model building codes require that standard tests be performed by an approved testing agency.

Special inspector(s) are responsible for the observation and verification of the testing procedures performed in the field. Special inspectors determine compliance with test standards based on their interpretation of the standards, as measured against acceptance criteria that are included in the construction documents and the quality assurance plan.

Test standards also assign responsibility to others. For example, the ASTM A 706 specification for low-alloy steel reinforcing bars requires the manufacturer to report the chemical composition and carbon equivalent of the material. In addition, the ANSI/AWS D1.4 Welding Code requires the contractor to prepare written specifications for the welding of reinforcing bars. It is necessary, therefore, that each member of the construction team has a thorough knowledge of the specified test standards that cover their particular work.

2.4.5 Mechanical and electrical equipment. The registered design professional should consider requirements to demonstrate the seismic performance of mechanical and electrical components critical to the post-earthquake life safety of the occupants. Any requirements should be clearly indicated on the construction documents. Any currently accepted technology should be acceptable to demonstrate compliance with the requirements.

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 the *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.

2.5 STRUCTURAL OBSERVATIONS

The purpose of structural observations is to allow the registered design professional(s) in responsible charge or other registered design professional(s) to visit the site to observe the seismic-force-resisting systems. Observations include verifying that the seismic-force-resisting system is constructed in general conformance with the construction documents, that the intent of the design has been accomplished, and that a complete lateral load path exists.

Every effort shall be made to have the registered design professional in responsible charge make the observations. If another registered design professional performs the observations he is expected to be familiar with the construction documents and the design concept.

2.6 REPORTING AND COMPLIANCE PROCEDURES

The purpose of this section is to keep key parties informed of the special inspector’s observations and the contractor’s corrections.

Chapter 3 Commentary

GROUND MOTION

3.1 GENERAL

3.1.3 Definitions. The *Provisions* are intended to provide uniform levels of performance for structures, depending on their occupancy and use and the risk to society inherent in their failure. Sec. 1.2 of the *Provisions* establishes a series of Seismic Use Groups, which are used to assign each structure to a specific Seismic Design Category. It is the intent of the *Provisions* that meeting the seismic design criteria will provide a uniform margin against failure for all structures within a given Seismic Use Group.

In past editions of the *Provisions*, seismic hazards around the nation were defined at a uniform 10 percent probability of exceedance in 50 years and the design requirements were based on assigning a structure to a Seismic Hazard Exposure Group and a Seismic Performance Category. While this approach provided for a uniform likelihood throughout the nation that the design ground motion would not be exceeded, it did not provide for a uniform probability of failure for structures designed for that ground motion. The reason for this is that the rate of change of earthquake ground motion versus likelihood is not constant in different regions of the United States.

The approach adopted in the *Provisions* is intended to provide for a uniform margin against collapse at the design ground motion. In order to accomplish this, ground motion hazards are defined in terms of maximum considered earthquake ground motions. The maximum considered earthquake ground motions are based on a set of rules that depend on the seismicity of an individual region. The design ground motions are based on a lower bound estimate of the margin against collapse inherent in structures designed to the *Provisions*. This lower bound was judged, based on experience, to correspond to a factor of about 1.5 in ground motion. Consequently, the design earthquake ground motion was selected at a ground shaking level that is 1/1.5 (2/3) of the maximum considered earthquake ground motion.

For most regions of the nation, the maximum considered earthquake ground motion is defined with a uniform probability of exceedance of 2 percent in 50 years (return period of about 2500 years). While stronger shaking than this could occur, it was judged that it would be economically impractical to design for such very rare ground motions and that the selection of the 2 percent probability of exceedance in 50 years as the maximum considered earthquake ground motion would result in acceptable levels of seismic safety for the nation.

In regions of high seismicity, such as coastal California, the seismic hazard is typically controlled by large-magnitude events occurring on a limited number of well-defined fault systems. Ground shaking calculated at a 2 percent probability of exceedance in 50 years would be much larger than that which would be expected based on the characteristic magnitudes of earthquakes on these known active faults. This is because these major active faults can produce characteristic earthquakes every few hundred years. For these regions, it is considered more appropriate to directly determine maximum considered earthquake ground motions based on the characteristic earthquakes of these defined faults. In order to provide for an appropriate level of conservatism in the design process, when this approach to calculation of the maximum considered earthquake ground motion is used, the median estimate of ground motion resulting for the characteristic event is multiplied by 1.5.

Sec. 4.1.1 of the *Provisions* defines the maximum considered earthquake ground motion in terms of the mapped values of the spectral response acceleration at short periods, S_S, and at 1 second, S_1, for Class B sites. These values may be obtained directly from Maps 1 through 24, respectively. A detailed explanation for the development of Maps 1 through 24 appears as Appendix A to this *Commentary* volume. The procedure by which these maps were created, as described above and in Appendix A, is also included in the *Provisions* under Sec 3.4 so that registered design professionals performing such studies may use methods consistent with those that served as the basis for developing the maps.

3.2 GENERAL REQUIREMENTS

3.2.2 Procedure selection. This section sets alternative procedures for determining ground shaking parameters for use in the design process. The design requirements generally use response spectra to represent ground motions in the design process. For the purposes of the *Provisions*, these spectra are permitted to be determined using either a generalized procedure in which mapped seismic response acceleration parameters are referred to or by site-specific procedures. The generalized procedure in which mapped values are used is described in Sec. 3.3. The site-specific procedure is described in Sec. 3.4.

3.3 GENERAL PROCEDURE

This section provides the procedure for obtaining design site spectral response accelerations using the maps provided with the *Provisions*. Many buildings and structures will be designed using the equivalent lateral force procedure of Sec. 5.2, and this general procedure to determine the design spectral response acceleration parameters, S_{DS} and S_{D1}, that are directly used in that procedure. Some structures will be designed using the response spectrum procedure of Sec. 5.3. This section also provides for the development of a general response spectrum, which may be used directly in the modal analysis procedure, from the design spectral response acceleration parameters, S_{DS} and S_{D1}.

Maps 1 and 2 respectively provide two parameters, S_S and S_1, based on a national seismic hazard study conducted by the U.S. Geological Survey. For most buildings and sites, they provide a suitably accurate estimate of the maximum considered earthquake ground shaking for design purposes. For some sites, with special soil conditions or for some buildings with special design requirements, it may be more appropriate to determine a site-specific estimate of the maximum considered earthquake ground shaking response accelerations. Sec. 3.4 provides guidance on site-specific procedures.

S_S is the mapped value, from Map 1 of the 5-percent-damped maximum considered earthquake spectral response acceleration, for short period structures founded on Class B, firm rock, sites. The short-period acceleration has been determined at a period of 0.2 seconds. This is because it was concluded that 0.2 seconds was reasonably representative of the shortest effective period of buildings and structures that are designed by the *Provisions*, considering the effects of soil compliance, foundation rocking, and other factors typically neglected in structural analysis.

Similarly, S_1 is the mapped value from Map 2 of the 5-percent-damped maximum considered earthquake spectral response acceleration at a period of 1 second on Site Class B. The spectral response acceleration at periods other than 1 second can typically be derived from the acceleration at 1 second. Consequently, these two response acceleration parameters, S_S and S_1, are sufficient to define an entire response spectrum for the period range of importance for most buildings and structures, for maximum considered earthquake ground shaking on Class B sites.

In order to obtain acceleration response parameters that are appropriate for sites with other characteristics, it is necessary to modify the S_S and S_1 values, as indicated in Sec.3.3.2. This modification is performed with the use of two coefficients, F_a and F_v, which respectively scale the S_S and S_1 values determined for firm rock sites to values appropriate for other site conditions. The maximum considered earthquake spectral response accelerations adjusted for Site Class effects are designated S_{MS} and S_{M1}, respectively, for short-period and 1-second-period response. As described above, structural design in the *Provisions* is performed for earthquake demands that are 2/3 of the maximum considered earthquake response spectra. Two additional parameters, S_{DS} and S_{D1}, are used to define the acceleration response spectrum for this design level event. These are taken, respectively, as 2/3 of the maximum considered earthquake values, S_{MS} and S_{M1}, and completely define a design response spectrum for sites of any characteristics.

Sec. 3.5.1 provides a categorization of the various classes of site conditions, as they affect the design response acceleration parameters. Sec. 3.5.2 describes the steps by which sites can be classified as belonging to one of these Site Classes.

3.3.2 Site coefficients and adjusted acceleration parameters. The site coefficients Fa and Fv presented in *Provisions* Tables 3.3-1 and 3.3-2 are based on the research described in the following paragraphs.

It has long been recognized that the effects of local soil conditions on ground motion characteristics should be considered in building design. The 1989 Loma Prieta earthquake provided abundant strong motion data that was used extensively together with other information in introducing the site coefficients Fa and Fv into the 1994 *Provisions.*

The amount of ground motion amplification by a soil deposit relative to bedrock depends on the wave-propagation characteristics of the soil, which can be estimated from measurements or inferences of shear-wave velocity and in turn the shear modulus for the materials as a function of the level of shaking. In general, softer soils with lower shear-wave velocities exhibit higher amplifications than stiffer soils with higher shear velocities. Increased levels of ground shaking result in increased soil stress-strain nonlinearity and increased soil damping which in general reduces the amplification, especially for shorter periods. Furthermore, for soil deposits of sufficient thickness, soil amplification is generally greater at longer periods than at the shorter periods. Based on the studies summarized below, values of the soil amplification factors (site coefficients) shown in Tables 3.3-1 and 3.3-2 were developed as a function of site class and level of ground shaking. Table 3.3-1 presents the short-period site coefficient, F_a; Table 3.3-2 presents the long-period site coefficient, F_v. As described in Sec. 3.5, Site Classes A through E describe progressively softer (lower shear wave velocity) soils.

Strong-motion recordings obtained on a variety of geologic deposits during the Loma Prieta earthquake of October 17, 1989 provided an important empirical basis for the development of the site coefficients F_a and F_v. Figure C3.3.2-1 presents average response spectra of ground motions recorded on soft clay and rock sites in San Francisco and Oakland during the Loma Prieta earthquake. The peak acceleration (which plots at zero-period of the response spectra) was about 0.08 to 0.1 g at the rock sites and was amplified two to three times to 0.2 g or 0.3 g at the soft soil sites. The response spectral accelerations at short periods (~ 0.2 or 0.3 second) were also amplified on average by factors of 2 or 3. It can be seen in Figure C3.3.2-1 that, at longer periods between about 0.5 and 1.5 or 2 seconds, the amplifications of response spectra on the soft clay site relative to rock were even greater, ranging from about 3 to 6 times. Ground motions on stiff soil sites were also observed to be amplified relative to rock sites during the Loma Prieta earthquake, but by smaller factors than on soft soils.

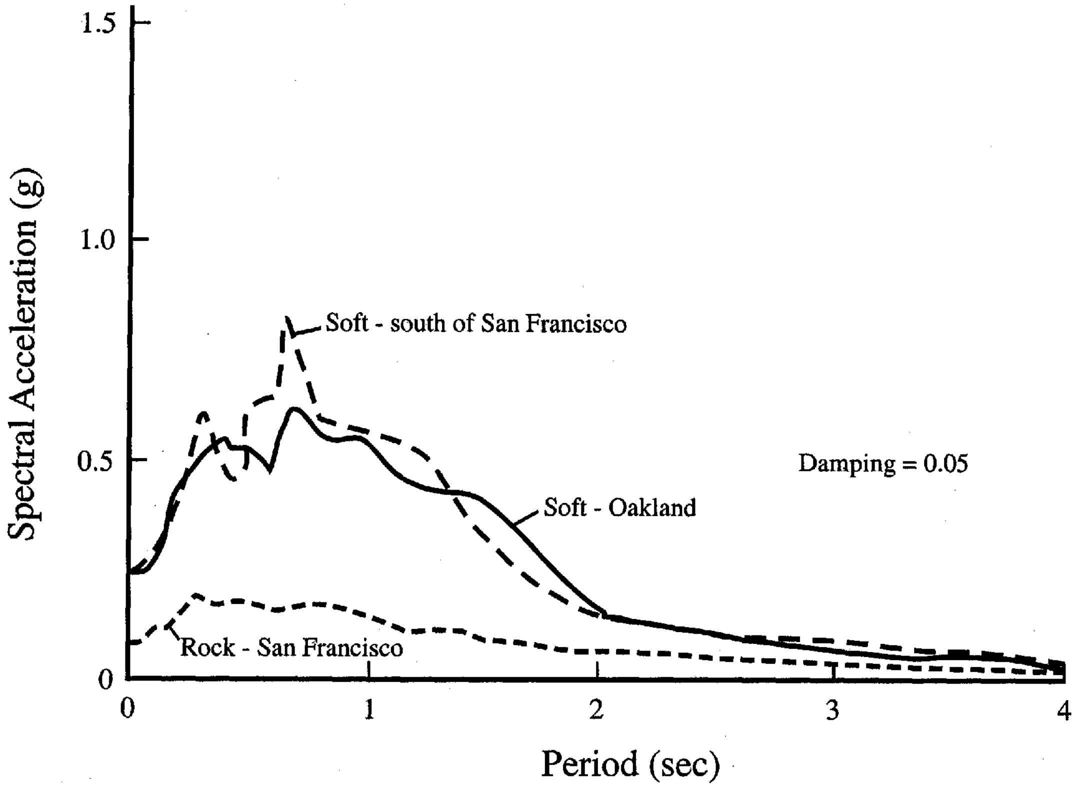


Figure C3.3.2-1. Average spectra recorded during the 1989 Loma Prieta earthquake in San Francisco Bay area at rock sites and soft soil sites (modified after Housner, 1990).

Average amplification factors derived from the Loma Prieta earthquake data with respect to “firm to hard rock” for short-period (0.1-0.5 sec), intermediate-period (0.5-1.4 sec), mid-period (0.4-2.0 sec), and long-period (1.5-5.0 sec) bands, show that a short-period factor and a mid-period factor (the mid-period factor was later renamed the long-period factor in the NEHRP Provisions) are sufficient to characterize the response of the local site conditions (Borcherdt, 1994). This important result is consistent with the two-factor approach to response spectrum construction summarized in Figure C3.3.2-2. Empirical regression curves fitted to these amplification data as a function of mean shear wave velocity at a site are shown in Figure C3.3.2-3.

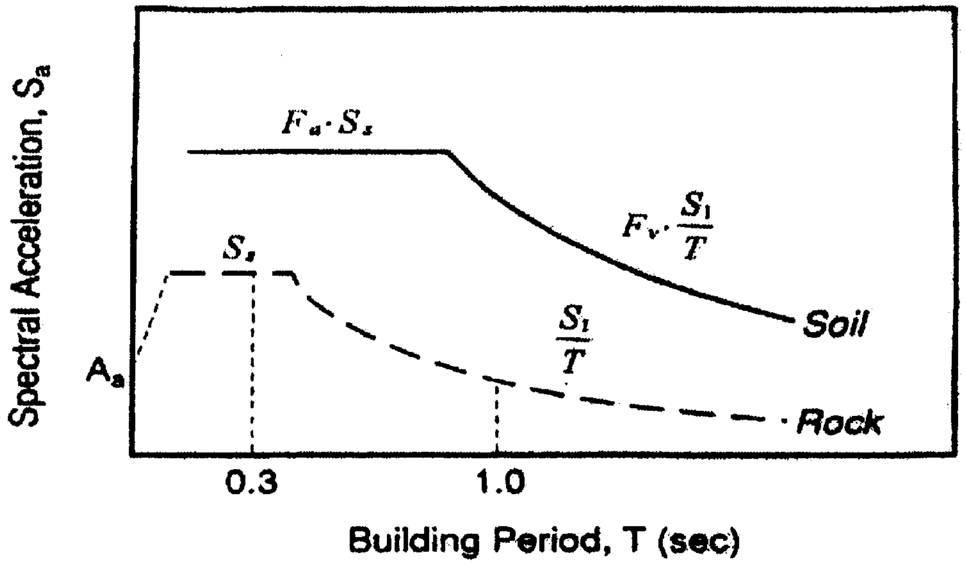


Figure C3.3.2-2. Two-factor approach to local site response.

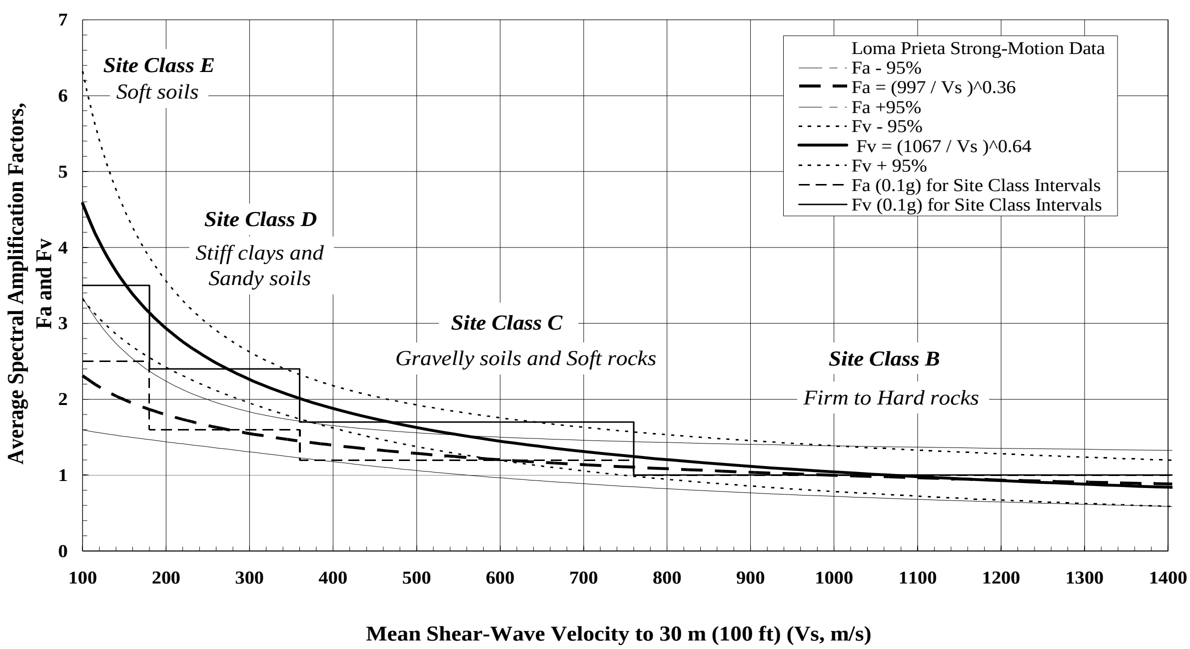


Figure C3.3.2-3. Short-period Fa and long-period Fv site coefficients with respect to site class B (firm to hard rocks) inferred as a continuous function of shear-wave velocity from empirical regression curves derived using Loma Prieta strong-motion recordings. The 95 percent confidence intervals for the ordinate to the true population regression line and the corresponding site coefficients in Tables 3.3-1 and 3.3-2 for 0.1g acceleration are plotted. The curves show that a two factor approach with a short- and a long-period site coefficient is needed to characterize the response of near surface deposits (modified from Borcherdt 1994).

The curves in Figure C3.3.2-3 provide empirical estimates of the site coefficients F_a and F_v as a function of mean shear wave velocity for an input peak ground accelerations on rock equal to about 0.1 g (Borcherdt, 1994; Borcherdt and Glassmoyer, 1994) The empirical amplification factors predicted by these curves are in good agreement with those obtained from empirical analyses of Loma Prieta data for soft soils by Joyner et al. (1994) shown in Figure 3.3.2-4. These short- and long-period amplification factors for low peak ground (rock) acceleration levels (~ 0.1 g) provided the basis for the values in the left-hand columns of Tables 3.3-1 and 3.3-2. Note that in Tables 3.3-1 and 3.3-2, a peak ground (rock) acceleration of 0.1g corresponds approximately to a response spectral acceleration on rock at 0.2-second period (S_s) equal to 0.25g (Table 3.3-1) and to a response spectral acceleration on rock at 1.0-second period (S_1) equal to 0.1g (Table 3.3-2).

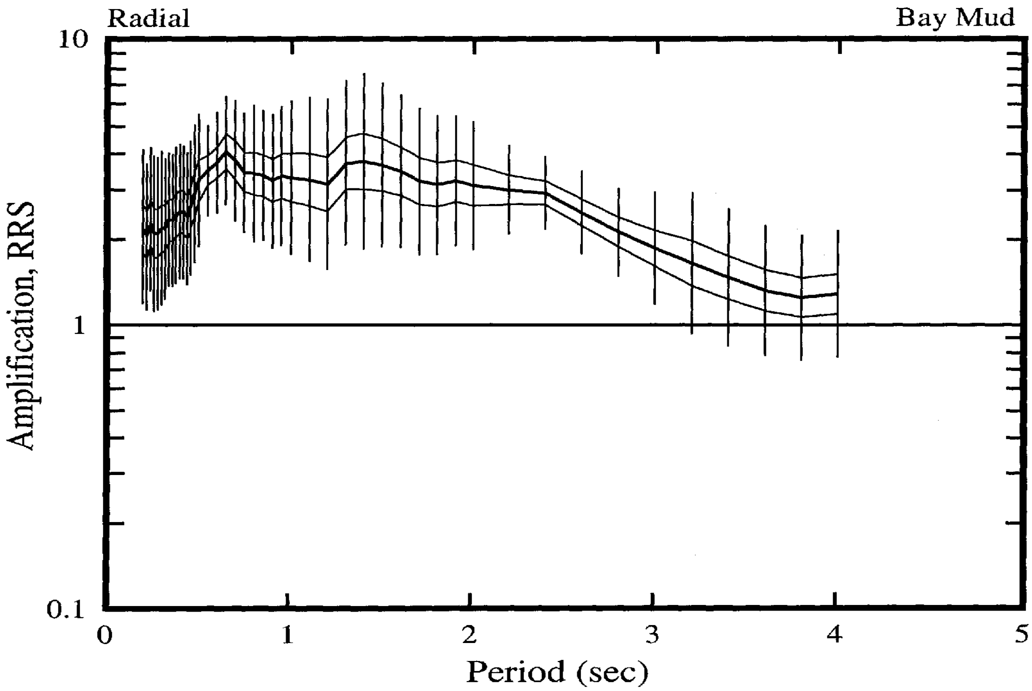


Figure C3.3.2-4. Calculation of average ratios of response spectra (RRS) curves for 5 percent damping from records of 1989 Loma Prieta earthquake on soft soil sites. The middle curve gives the geometric average ratio as function of the period. The top and bottom curves show the range from plus to minus one standard deviation of the average of the logarithms of the ratios. The vertical lines show the range from plus to minus standard deviation of the logarithms of the ratios (Joyner et al., 1994).

The values of F_a and F_v obtained directly from the analysis of ground motion records from the Loma Prieta earthquake were used to calibrate numerical one-dimensional site response analytical techniques, including equivalent linear as well as nonlinear programs. The equivalent linear program SHAKE (Schnabel et al. 1972), which had been shown in previous studies to provide reasonable predictions of soil amplification during earthquakes (e.g., Seed and Idriss 1982), was used extensively for this calibration. Seed et al. (1994) and Dobry et al. (1994) showed that the one-dimensional model provided a good first-order approximation to the observed site response in the Loma Prieta earthquake, especially at soft clay sites. Idriss (1990, 1991) used these analysis techniques to study the amplification of peak ground acceleration on soft soil sites relative to rock sites as a function of the peak acceleration on rock. Results of these studies are shown in Figure 3.3.2-5, illustrating that the large amplifications of peak acceleration on soft soil for low rock accelerations recorded during the 1985 Mexico City earthquake and the 1989 Loma Prieta earthquake should tend to decrease rapidly as rock accelerations increases above about 0.1 g.

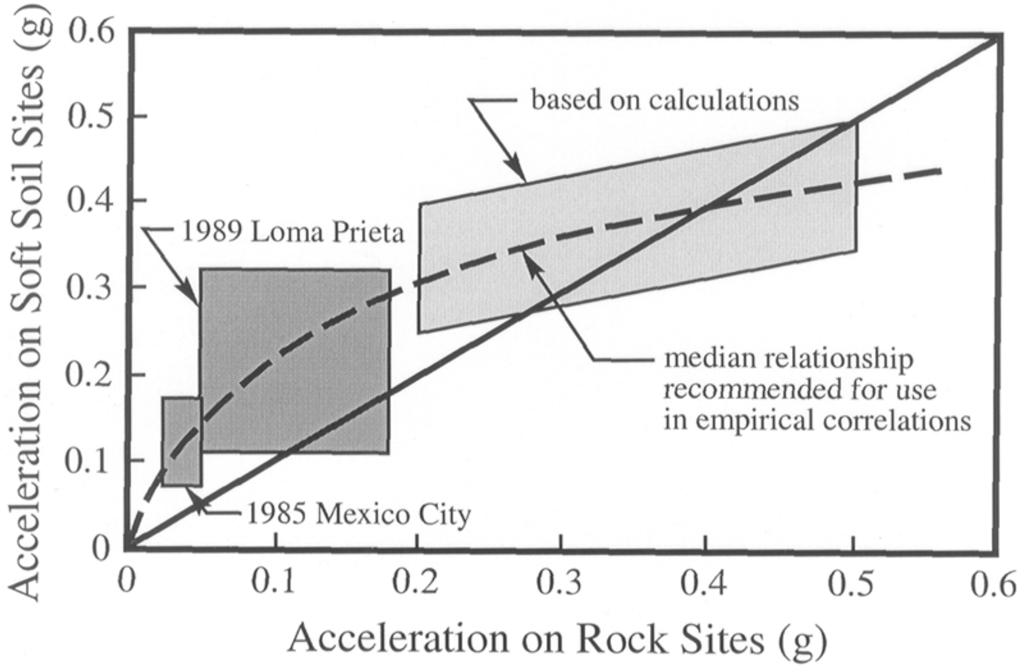
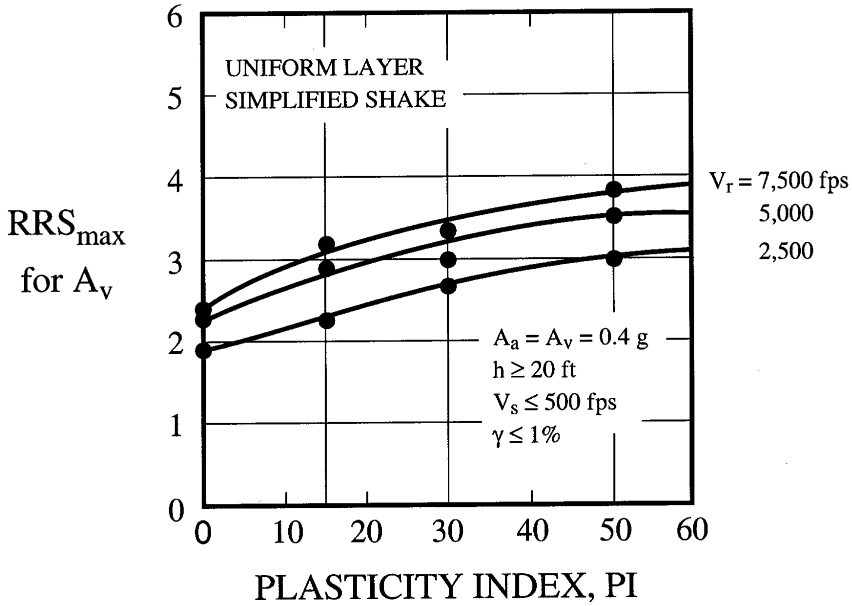


Figure C3.3.2-5. Relationships between maximum acceleration on rock and other local site conditions (Idriss, 1990, 1991).

After calibration, these equivalent linear and nonlinear one-dimensional site response techniques were used to extrapolate the values of F_a and F_v to larger rock accelerations of as much as 0.4g or 0.5g. Parametric studies involving combinations of hundreds of soil profiles and several dozen input earthquake rock motions provided quantitative guidelines for extrapolation of the Loma Prieta earthquake results (Seed et al. 1994; Dobry et al. 1994). Figure C3.3.2-6 summarizes some results of these site response analyses using the equivalent linear method. This figure presents values of peak amplification of response spectra at long periods for soft sites (termed maximum Ratio of Response Spectra, RRS_{max}) calculated using the equivalent linear approach as a function of the plasticity index (PI) of the soil and the rock shear wave velocity V_r for both weak (0.1 g) and strong (0.4 g) input rock shaking. The effect of PI is due to the fact that soils with higher PI exhibit less stress-strain nonlinearity and a lower material damping (Vucetic and Dobry 1991). For peak rock acceleration = 0.1 g, V_r = 4,000 ft/sec (1220 m/s) and PI = 50, roughly representative of San Francisco Bay area soft sites in the Loma Prieta earthquake, RRS_{max} = 4.4, which for a soil shear wave velocity of 150 m/sec coincides with the upper part of the range in Figure 3.3.2-3 inferred from the ground motion records. Note the reduction of this value of RRS_{max} from 4.4 to about 3.3 in Figure C3.3.2-6 when peak rock acceleration = 0.4 g, due to soil nonlinearity. Results such as those in Figure C3.3.2-6 provided the basis for the values of F_a and F_v shown in the right-most four columns of Tables 3.3-1 and 3.3-2.



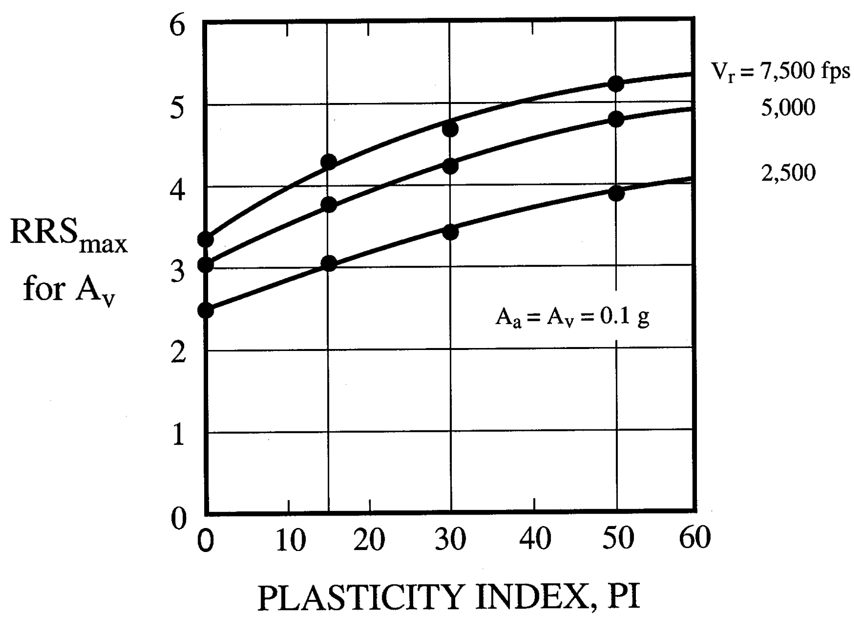
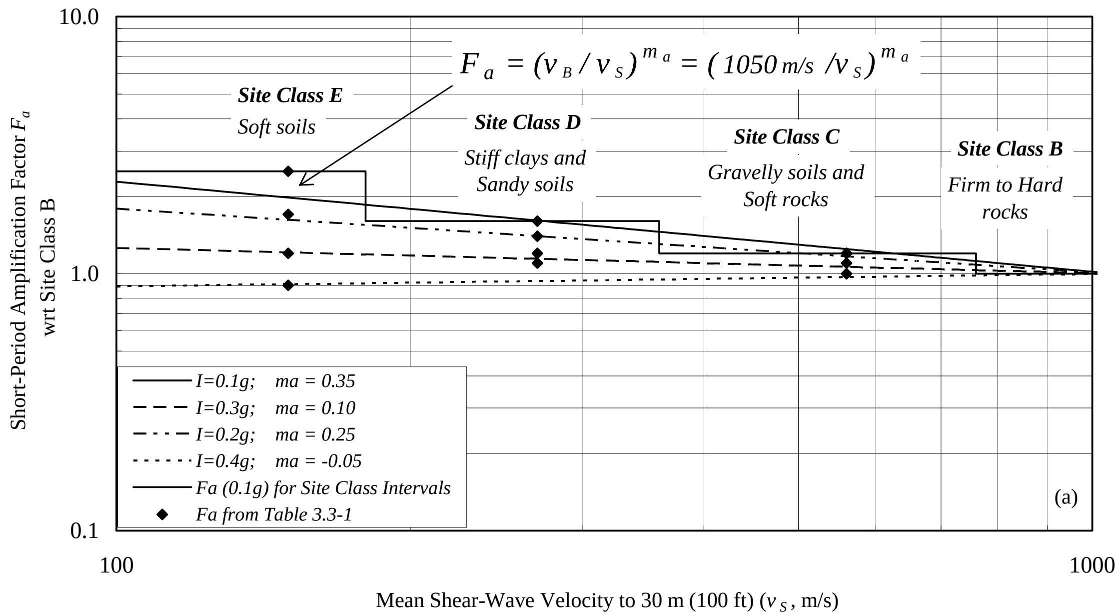


Figure C3.3.2-6. Variation of RRSmax of uniform layer of soft clay on rock from equivalent linear site response analyses (Dobry et al., 1994).

Graphs and equations that provide a framework for extrapolation of F_a and F_v from Loma Prieta results to larger input ground motion levels continuously as a function of site conditions (shear-wave velocity) are shown in Figure C3.3.2-7. The site coefficients in Tables 3.3-1 and 3.3-2 are superimposed on this figure. These simple curves were developed to reproduce the site coefficients for site classes E and B and provide approximate estimates of the coefficients for the other Site Classes at various ground acceleration levels. The equations describing the curves indicate that the amplification at a site is proportional to the shear velocity ratio (impedance ratio) with an exponent that varies with the input ground motion level (Borcherdt, 1994).



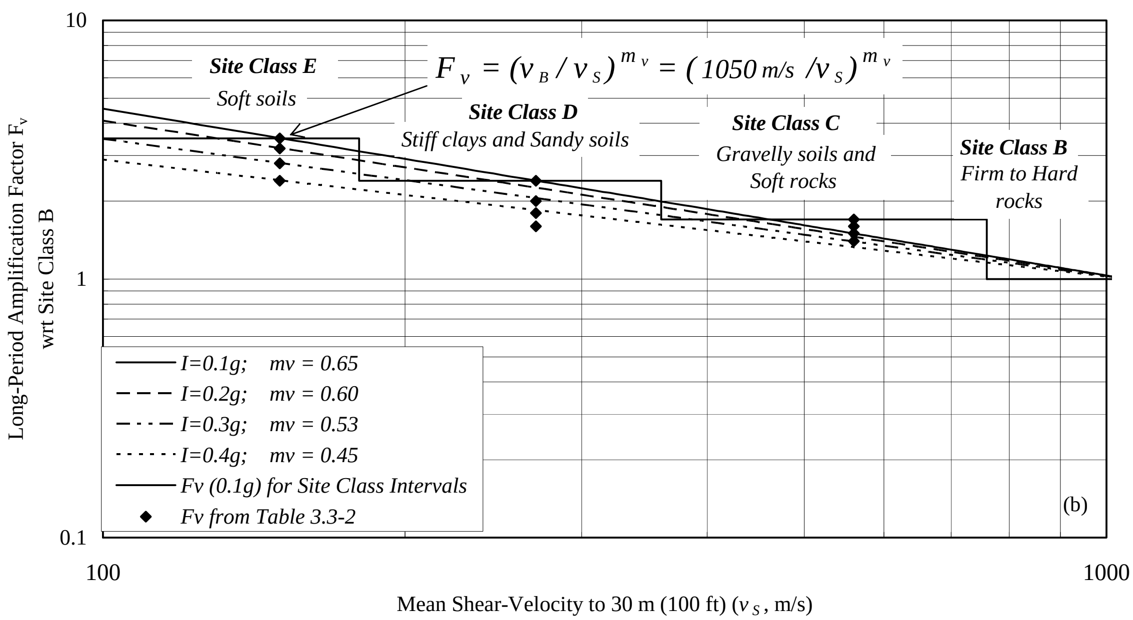


Figure C3.3.2-7. Graphs and equations that provide a simple framework for inference of (a) F_a and (b) F_v values as a continuous function of shear velocity at various input acceleration levels. Site coefficients in Table 3.3-1 and 3.3-2 are superimposed. These simple curves were developed to reproduce the site coefficients for site classes E and B and provide approximate estimates of the coefficients for the other site classes at various ground acceleration levels (from Borcherdt 1994).

A more extensive discussion of the development of site coefficients is presented by Dobry, et al. (2000). Since the development of these coefficients and the development of a community consensus regarding their values in 1992, recent earthquakes have provided additional strong motion data from which to infer site amplifications. Analyses conducted on the basis of these more recent data are reported by a number of researchers, including Crouse and McGuire, 1996; Dobry et al., 1999; Silva et al., 2000; Joyner and Boore, 2000; Field, 2000; Steidl, 2000; Rodriquez-Marek et al., 2001; Borcherdt, 2002, and Stewart et al., 2003. While the results of these studies vary, overall the site amplification factors are generally consistent with those in Tables 3.3-1 and 3.3.2 and there is no clear consensus for change at the present time (end of 2002).

3.3.4 Design response spectrum. This section provides a general method for obtaining a 5-percent-damped response spectrum from the site design acceleration response parameters S_{aS} and S_{a1}. This spectrum is based on that proposed by Newmark and Hall, as a series of three curves representing in the short period, a region of constant spectral response acceleration; in the long period, a range of constant spectral response velocity; and in the very long period, a range of constant spectral response displacement. Response acceleration at any period in the long period range can be related to the constant response velocity by the equation:

$$S_a = \omega S_v = \frac{2\pi}{T} S_v \tag{C3.3-1}$$

where ω is the circular frequency of motion, T is the period, and S_v is the constant spectral response velocity. Thus the site design spectral response acceleration at 1 second, S_{a1}, is simply related to the constant spectral velocity for the spectrum as follows:

$$S_{a1} = 2\pi S_v \tag{C3.3-2}$$

and the spectral response acceleration at any period in the constant velocity range can be obtained from the relationship:

$$S_a = \frac{S_{D1}}{T} \tag{C3.3-3}$$

The constant displacement domain of the response spectrum is not included on the generalized response spectrum because relatively few structures have a period long enough to fall into this range. Response accelerations in the constant displacement domain can be related to the constant displacement by a $1/T^2$ relationship. Sec. 5.3 of the *Provisions*, which provides the requirements for modal analysis also provides instructions for obtaining response accelerations in the very long period range.

The T_L maps were prepared following a two-step procedure. The first step consisted of establishing a correlation between earthquake magnitude and T_L. This correlation was established by (1) determining the corner period between intermediate and long period motions based on seismic source theory and (2) examining the response spectra of strong motion accelerograms recorded during moderate and large magnitude earthquakes. This corner period, T_c, marks the transition between the constant displacement and constant velocity segments of the Fourier spectrum representing a theoretical fault-rupture displacement history. T_c, which was considered an approximation for T_L, was related to moment magnitude, M, through the formula, $\log T_c = -1.25 + 0.3 M$. This formula was selected from several available formulas based on comparisons of T_c predicted by this equation and T_L estimated from strong motion accelerograms with reliable long period content. The results were used to establish the following half-unit ranges of M for given values of T_c..

M	T_c (sec)
6.0 – 6.5	4
6.5 – 7.0	6
7.0 – 7.5	8
7.5 – 8.0	12
8.0 – 8.5	16
8.5 – 9.0+	20

To determine the T_L values for the U.S., the USGS constructed maps of the modal magnitudes (M_d) in half-unit increments (as shown in the above table). The maps were prepared from a deaggregation of the 2 percent in 50-yr hazard for S_a (T = 2 sec), the response spectral acceleration at an oscillator period of 2 sec. (for HI the deaggregation was only available for T = 1 sec). The M_d that was computed represented the magnitude interval that had the largest contribution to the 2 percent in 50-yr hazard for S_a.

The M_d maps were judged to be an acceptable approximation to values of M_d that would be obtained if the deaggregation could have been computed at the longer periods of interest. These M_d maps were color coded to more easily permit the eventual construction of the T_L maps. Generally the T_L maps corresponded to the M_d maps, but some smoothing of the boundaries separating T_L regions was necessary to make them more legible. A decision was made to limit the T_L in the broad area in the central and eastern U.S., which had an M_d of 16 sec, to 12 sec, Likewise, the T_L for the areas affected by the great megathrust earthquakes in the Pacific Northwest and Alaska, was limited to 16 sec.

3.4 SITE-SPECIFIC PROCEDURE

The objective in conducting a site-specific ground motion analysis is to develop ground motions that are determined with higher confidence for the local seismic and site conditions than can be determined from national ground motion maps and the general procedure of Sec. 3.3. Accordingly, such studies must be comprehensive and incorporate current scientific interpretations. Because there is typically more than one scientifically credible alternative for models and parameter values used to characterize seismic sources and ground motions, it is important to formally incorporate these uncertainties in a site-specific probabilistic analysis. For example, uncertainties may exist in seismic source location, extent and geometry; maximum earthquake magnitude; earthquake recurrence rate; choices for ground motion attenuation relationships; and local site conditions including soil layering and dynamic soil properties as well as possible two- or three-dimensional wave propagation effects. The use of peer review for a site-specific ground motion analysis is encouraged.

Near-fault effects on horizontal response spectra include (1) directivity effects that increase ground motions for periods of vibration greater than approximately 0.5 second for fault rupture propagating toward the site; and (2) directionality effects that increase ground motions for periods greater than approximately 0.5 second in the direction normal (perpendicular) to the strike of the fault. Further discussion of these effects is contained in Somerville et al. (1997) and Abrahamson (2000).

Conducting site-specific geotechnical investigations and dynamic site response analyses.

Provisions Tables 3.3-1 and 3.3-2 and Sec. 3.5.1 require that site-specific geotechnical investigations and dynamic site response analysis be performed for sites having Site Class F soils. Guidelines are provided below for conducting site-specific investigations and site response analyses for these soils. These guidelines are also applicable if it is desired to conduct dynamic site response analyses for other site classes.

Site-specific geotechnical investigation: For purposes of obtaining data to conduct a site response analysis, site-specific geotechnical investigations should include borings with sampling, standard penetration tests (SPTs) for sandy soils, cone penetrometer tests (CPTs), and/or other subsurface investigative techniques and laboratory soil testing to establish the soil types, properties, and layering and the depth to rock or rock-like material. For very deep soil sites, the depth of investigation need not necessarily extend to bedrock but to a depth that may serve as the location of input motion for a dynamic site response analysis (see below). It is desirable to measure shear wave velocities in all soil layers. Alternatively, shear wave velocities may be estimated based on shear wave velocity data available for similar soils in the local area or through correlations with soil types and properties. A number of such correlations are summarized by Kramer (1996).

Dynamic site response analysis: Components of a dynamic site response analysis include the following steps:

1. Modeling the soil profile: Typically, a one-dimensional soil column extending from the ground surface to bedrock is adequate to capture first-order site response characteristics. For very deep soils, the model of the soil columns may extend to very stiff or very dense soils at depth in the column. Two- or three-dimensional models should be considered for critical projects when two or three-dimensional wave propagation effects should be significant (e.g., in basins). The soil layers in a one-dimensional model are characterized by their total unit weights and shear wave velocities from which low-strain (maximum) shear moduli may be obtained, and by relationships defining the nonlinear shear stress-strain relationships of the soils. The required relationships for analysis are often in the form of curves that describe the variation of soil shear modulus with shear strain (modulus reduction curves) and by curves that describe the variation of soil damping with shear strain (damping curves). In a two- or three-dimensional model, compression wave velocities or moduli or Poisson ratios also are required. In an analysis to estimate the effects of liquefaction on soil site response, the nonlinear soil model also must incorporate the buildup of soil pore water pressures and the consequent effects on reducing soil stiffness and strength. Typically, modulus reduction curves and damping curves are selected on the basis of published relationships for similar soils (e.g., Seed and Idriss, 1970; Seed et al., 1986; Sun et al., 1988; Vucetic and Dobry, 1991; Electric Power Research Institute, 1993; Kramer, 1996). Site-specific laboratory dynamic tests on soil samples to establish nonlinear soil characteristics can be considered where published relationships are judged to be inadequate for the types of soils present at the site. Shear and compression wave velocities and associated maximum moduli should be selected on the basis of field tests to determine these parameters or published relationships and experience for similar soils in the local area. The uncertainty in soil properties should be estimated, especially the uncertainty in the selected maximum shear moduli and modulus reduction and damping curves.

2. Selecting input rock motions: Acceleration time histories that are representative of horizontal rock motions at the site are required as input to the soil model. Unless a site-specific analysis is carried out to develop the rock response spectrum at the site, the maximum considered earthquake (MCE) rock spectrum for Site Class B rock can be defined using the general procedure described in Sec. 3.3. For hard rock (Site Class A), the spectrum may be adjusted using the site factors in Tables 3.3-1 and 3.3-2. For profiles having great depths of soil above Site Class A or B rock, consideration can be given to defining the base of the soil profile and the input rock motions at a depth at which soft rock or very stiff soil of Site Class C is encountered. In such cases, the MCE rock response spectrum may be taken as the spectrum for Site Class C defined using the site factors in Tables 3.3-1 and 3.3-2. Several acceleration time histories of rock motions, typically at least four, should be selected for site response analysis. These time histories should be selected after evaluating the types of earthquake sources, magnitudes, and distances that predominantly contribute to the seismic hazard at the site. Preferably, the time histories selected for analysis should have been recorded on geologic materials similar to the site class of materials at the base of the site soil profile during earthquakes of similar types (e.g. with respect to tectonic environment and type of faulting), magnitudes, and distances as those predominantly contributing to the site

seismic hazard. The U.S. Geological Survey national seismic hazard mapping project website (http://geohazards.cr.usgs.gov/eq/) includes hazard deaggregation options and can be used to evaluate the predominant types of earthquake sources, magnitudes, and distances contributing to the hazard. Sources of recorded acceleration time histories include the data bases of the Consortium of Organizations for Strong Motion Observation Systems (COSMOS) Virtual Data Center web site (db.cosmos-eq.org) and the Pacific Earthquake Engineering Research Center (PEER) Strong Motion Data Base website (http://peer.berkeley.edu/smcat/). Prior to analysis, each time history should be scaled so that its spectrum is at the approximate level of the MCE rock response spectrum in the period range of interest. It is desirable that the average of the response spectra of the suite of scaled input time histories be approximately at the level of the MCE rock response spectrum in the period range of interest. Because rock response spectra are defined at the ground surface rather than at depth below a soil deposit, the rock time histories should be input in the analysis as outcropping rock motions rather than at the soil-rock interface.

3. Site response analysis and results interpretation: Analytical methods may be equivalent linear or nonlinear. Frequently used computer programs for one-dimensional analysis include the equivalent linear program SHAKE (Schnabel et al., 1972; Idriss and Sun, 1992) and the nonlinear programs DESRA-2 (Lee and Finn, 1978), MARDES (Chang et al., 1991), SUMDES (Li et al., 1992), D-MOD (Matasovic, 1993), TESS (Pyke, 1992), and DESRAMUSC (Qiu, 1998). If the soil response is highly nonlinear (e.g., high acceleration levels and soft soils), nonlinear programs may be preferable to equivalent linear programs. For analysis of liquefaction effects on site response, computer programs incorporating pore water pressure development (effective stress analyses) must be used (e.g., DESRA-2, SUMDES, D-MOD, TESS, and DESRAMUSC). Response spectra of output motions at the ground surface should be calculated and the ratios of response spectra of ground surface motions to input outcropping rock motions should be calculated. Typically, an average of the response spectral ratio curves is obtained and multiplied by the MCE rock response spectrum to obtain a soil response spectrum. Sensitivity analyses to evaluate effects of soil property uncertainties should be conducted and considered in developing the design response spectrum.

3.4.2 Deterministic maximum considered earthquake. It is required that ground motions for the deterministic maximum considered earthquake be based on characteristic earthquakes on all known active faults in a region. As defined in Sec. 3.1.3, the magnitude of a characteristic earthquake on a given fault should be a best-estimate of the maximum magnitude capable for that fault but not less than the largest magnitude that has occurred historically on the fault. The maximum magnitude should be estimated considering all seismic-geologic evidence for the fault, including fault length and paleoseismic observations. For faults characterized as having more than a single segment, the potential for rupture of multiple segments in a single earthquake should be considered in assessing the characteristic maximum magnitude for the fault.

3.5 SITE CLASSIFICATION FOR SEISMIC DESIGN

3.5.1 Site Class Definitions. Based on the studies and observations discussed in Sec. 3.3-2, the site categories in the 2003 *Provisions* are defined in terms of the small-strain shear wave velocity in the top 100 ft (30 m) of the profile, $\bar{v}_s$, as might be inferred from travel time for a shear wave to travel from the surface to a depth of 100 ft (30m). If shear wave velocities are available for the site, they should be used to classify the site.

However, in recognition of the fact that in many cases the shear wave velocities are not available, alternative definitions of the site classes also are included in the 2003 *Provisions*. They use the standard penetration resistance for cohesionless and cohesive soils and rock and the undrained shear strength for cohesive soils only. These alternative definitions are rather conservative since the correlation between site amplification and these geotechnical parameters is more uncertain than the correlation with $\bar{v}_s$. That is, there will be cases where 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 site category definitions should not be interpreted as implying any specific numerical correlation between shear-wave velocity and standard penetration resistance or shear strength.

Equation 3.5-1 is for inferring the average shear-wave velocity to a depth of 100 ft (30m) at a site. Equation 3.5-1specifies that the average velocity is given by the sum of the thicknesses of the geologic layers in the upper 100 ft divided by the sum of the times for a shear wave to travel through each layer, where travel time for each layer is specified by the ratio of the thickness and the shear wave velocity for the layer. It is important that this method of averaging be used as it may result in a significantly lower effective average shear wave velocity than the velocity that would be obtained by averaging the velocities of the individual layers directly.

Equation 3.5-2 is for classifying the site using the standard penetration resistance (*N*-value) for cohesionless soils, cohesive soils, and rock in the upper 100 ft (30 m). A method of averaging analogous to the method of Equation 3.5-1 for shear wave velocity is used. The maximum value of *N* that can be used for any depth of measurement in soil or rock is 100 blows/ft.

Equations 3.5-3 and 3.5-4 are for classifying the site using the standard penetration resistance of cohesionless soil layers, N_{ch}, and the undrained shear strength of cohesive soil layers, s_u, within the top 100 ft (30 m). These equations are provided as an alternative to using Eq. 3.5-2 for which *N*-values in all geologic materials in the top 100 ft (30 m) are used. When using Eq. 3.5-3 and 3.5-4, only the thicknesses of cohesionless soils and cohesive soils within the top 100 ft (30 m) are used.

As indicated in Sec. 3.3-2 and 3.5-1, soils classified as Site Class F according to the definitions in Sec. 3.5-1 require site-specific evaluations. An exception is made, however, for liquefiable sites where the structure has a fundamental period of vibration equal to or less than 0.5 second. For such structures, values of F_a and F_v for the site may be determined using the site class definitions and criteria in Sec. 3.5-1 assuming liquefaction does not occur. The exception is provided because ground motion data obtained in liquefied soil areas during earthquakes indicate that short-period ground motions are generally attenuated due to liquefaction whereas long-period ground motions may be amplified. This exception is only for the purposes of defining the site class and obtaining site coefficients. It is still required to assess liquefaction potential and its effects on structures as a ground failure hazard as specified in Chapter 7.

3.5.2 Steps for classifying a site. A step- by- step procedure for classifying a site is given in the *Provisions*. Although the procedure and criteria in Sec. 3.5.1 and 3.5.2 are straightforward, there are aspects of these assessments that may require additional judgment and interpretation. Highly variable subsurface conditions beneath a building footprint could result in overly conservative or unconservative site classification. Isolated soft soil layers within an otherwise firm soil site may not affect the overall site response if the predominant soil conditions do not include such strata. Conversely, site response studies have shown that continuous, thin, soft clay strata may increase the site amplification.

The site class should reflect the soil conditions that will affect the ground motion input to the structure or a significant portion of the structure. For structures receiving substantial ground motion input from shallow soils (e.g. structures with shallow spread footings, laterally flexible piles, or structures with basements where it is judged that substantial ground motion input to the structure may come through the side walls), it is reasonable to classify the site on the basis of the top 100 ft (30 m) of soils below the ground surface. Conversely, for structures with basements supported on firm soils or rock below soft soils, it is reasonable to classify the site on the basis of the soils or rock below the mat, if it can be justified that the soft soils contribute very little to the response of the structure.

Buildings on sloping bedrock sites and/or having highly variable soil deposits across the building area require careful study since the input motion may vary across the building (for example, if a portion of the building is on rock and the rest is over weak soils). Site-specific studies including two- or three-dimensional modeling may be appropriate in such cases to evaluate the subsurface conditions and site and superstructure response. Other conditions that may warrant site-specific evaluation include the presence of low shear wave velocity soils below a depth of 100 ft (30 m), location of the site near the edge of a filled-in basin, or other subsurface or topographic conditions with strong two- and three-dimensional site-response effects. Individuals with appropriate expertise in seismic ground motions should participate in evaluations of the need for and nature of such site-specific studies.

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Chapter 4 Commentary

STRUCTURAL DESIGN CRITERA

4.1 GENERAL

4.1.2 References. 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.

4.2 GENERAL REQUIREMENTS

4.2.1 Design basis. Structural design for acceptable seismic resistance includes:

1. The selection of gravity- and seismic-force-resisting systems that are appropriate to the anticipated intensity of ground shaking;
2. Layout of these systems such that they provide a continuous, regular, and redundant load path capable of ensuring that the structures act as integral units in responding to ground shaking; and
3. Proportioning the various members and connections such that adequate lateral and vertical strength and stiffness is present to limit damage in a design earthquake to acceptable levels.

In the *Provisions*, the proportioning of structural elements (sizing of individual members, connections, and supports) is typically based on the distribution of internal forces computed based on linear elastic response spectrum analyses using response spectra that are representative of, but substantially reduced from the anticipated design ground motions. As a result, under the severe levels of ground shaking anticipated for many regions of the nation, the internal forces and deformations produced in most structures will substantially exceed the point at which elements of the structures start to yield or buckle and behave in an inelastic manner. This approach can be taken because historical precedent and the observation of the behavior of structures that have been subjected to earthquakes in the past demonstrates that if suitable structural systems are selected and structures are detailed with appropriate levels of ductility, regularity, and continuity, it is possible to perform an elastic design of structures for reduced forces and still achieve acceptable performance. Therefore, these procedures adopt the approach of proportioning structures such that under prescribed design lateral forces that are significantly reduced, by the response modification coefficient R, from those that would actually be produced by a design earthquake they will not deform beyond a point of significant yield. The elastic deformations calculated under these reduced design forces are then amplified, by the deflection amplification factor C_d to estimate the expected deformations likely to be experienced in response to the design ground motion. (Use of the deflection amplification factor is specified in Sec. 5.2.6.1.) Considering the intended structural performance and acceptable deformation levels, Sec. 4.5.1 prescribes the story drift limits for the expected (amplified) deformations. These procedures differ from those in earlier codes and design provisions wherein the drift limits were treated as a serviceability check.

The term “significant yield” is not the point where first yield occurs in any member but, rather, is defined as that level causing complete plastification of at least the most critical region of the structure (such as formation of a first plastic hinge in the structure). A structural steel frame comprising compact members is assumed to reach this point when a “plastic hinge” develops in the most highly stressed member of the structure. A concrete frame reaches significant yield when at least one of the sections of its most highly stressed component reaches its strength as set forth in Chapter 9. These requirements contemplate that the design includes a seismic-force-resisting system with redundant characteristics wherein significant structural overstrength above the level of significant yield can be obtained by plastification at other points in the structure prior to the formation of a complete mechanism. For example, Figure C4.2-1 shows the lateral load-deflection curve for a typical structure. Significant yield is the level where plastification occurs

at the most heavily loaded element in the structure, shown as the lowest yield hinge on the load-deflection diagram. With increased loading, causing the formation of additional plastic hinges, the capacity increases (following the solid curve) 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 design ground motion.

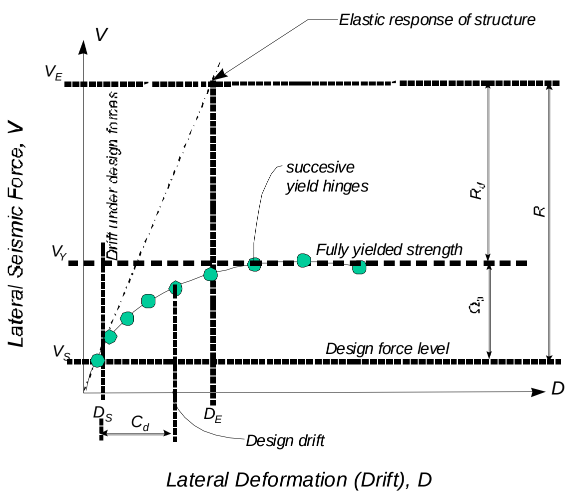

Figure C4.2-1 Inelastic force-deformation curve.

It should be noted that the structural overstrength described above results from the development of sequential plastic hinging in a properly designed, redundant structure. Several other sources will further increase structural overstrength. First, material overstrength (that is, actual material strengths higher than the nominal material strengths specified in the design) may increase the structural overstrength significantly. For example, a recent survey shows that the mean yield strength of A36 steel is about 30 to 40 percent higher than the minimum specified strength, which is used in design calculations. Second, member design strengths usually incorporate a strength reduction (or resistance) factor, ϕ, to ensure a low probability of failure under design loading. Third, designers themselves introduce additional overstrength by selecting sections or specifying reinforcing patterns that exceed those required by the computations. Similar situations occur when minimum requirements of the *Provisions*, for example, minimum reinforcement ratios, control the design. Finally, the design of many flexible structural systems, such as moment resisting frames, are often controlled by the drift rather than strength limitations of the *Provisions*, with sections selected to control lateral deformations rather than provide the specified strength. The results is that structures typically have a much higher lateral resistance than specified as a minimum by the *Provisions* and first actual significant yielding of structures may occur at lateral load levels that are 30 to 100 percent higher than the prescribed design seismic forces. If provided with adequate ductile detailing, redundancy, and regularity, full yielding of structures may occur at load levels that are two to four times the prescribed design force levels.

Figure C4.2-1 indicates the significance of design parameters contained in the *Provisions* including the response modification coefficient, R, the deflection amplification factor, C_d, and the structural overstrength coefficient Ω_0. The values of the response modification coefficient, R, structural overstrength coefficient, Ω_0, and the deflection amplification factor, C_d, provided in Table 4.3-1, as well as the criteria for story drift, including P-delta effects, have been established considering the characteristics of typical properly designed structures. If excessive "optimization" of a structural design is performed, with lateral resistance provided by only a few elements, the successive yield hinge behavior depicted in Figure C4.2-1 will not be able to form and the values of the design parameters contained in the *Provisions* may not be adequate to provide the intended seismic performance.

The response modification coefficient, R, 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 (see Figure C4.2-1). The structure is to be designed so that the level of significant yield exceeds the prescribed design force. The ratio R, expressed by the equation:

$$R = \frac{V_E}{V_S} \quad \text{(C4.2-1)}$$

is always larger then 1.0; thus, all structures are designed for forces smaller than those the design ground motion would produce in a structure with completely linear-elastic response. This reduction is possible for a number of reasons. As the structure begins to yield and deform inelastically, the effective period of response of the structure tends to lengthen, which for many structures, results in a reduction in strength demand. Furthermore, the inelastic action results in a significant amount of energy dissipation, also known as hysteretic damping, in addition to the viscous damping. The combined effect, which is also known as the ductility reduction, explains why a properly designed structure with a fully yielded strength (V_y in Figure C4.2-1) that is significantly lower than the elastic seismic force demand (V_E in Figure C4.2-1) can be capable of providing satisfactory performance under the design ground motion excitations. Defining a system ductility reduction factor R_d as the ratio between V_E and V_Y (Newmark and Hall, 1981):

$$R_d = \frac{V_E}{V_Y} \quad \text{(C4.2-2)}$$

then it is clear from Figure C4.2-1 that the response modification coefficient, R, is the product of the ductility reduction factor and structural overstrength factor (Uang, 1991):

$$R = R_d \Omega_0 \quad \text{(C4.2-3)}$$

The energy dissipation resulting from hysteretic behavior can be measured as the area enclosed by the force-deformation curve of the structure as it experiences several cycles of excitation. Some structures have far more energy dissipation capacity than do others. The extent of energy dissipation capacity available is largely dependent on the amount of stiffness and strength degradation the structure undergoes as it experiences repeated cycles of inelastic deformation. Figure C4.2-2 indicates representative load-deformation curves for two simple substructures, such as a beam-column assembly in a frame. Hysteretic curve (a) in the figure is representative of the behavior of substructures that have been detailed for ductile behavior. The substructure can maintain nearly all of its strength and stiffness over a number of large cycles of inelastic deformation. The resulting force-deformation "loops" are quite wide and open, resulting in a large amount of energy dissipation capacity. Hysteretic curve (b) represents the behavior of a substructure that has not been detailed for ductile behavior. It rapidly loses stiffness under inelastic deformation and the resulting hysteretic loops are quite pinched. The energy dissipation capacity of such a substructure is much lower than that for the substructure (a). Structural systems with large energy dissipation capacity have larger R_d values, and hence are assigned higher R values, resulting in design for lower forces, than systems with relatively limited energy dissipation capacity.

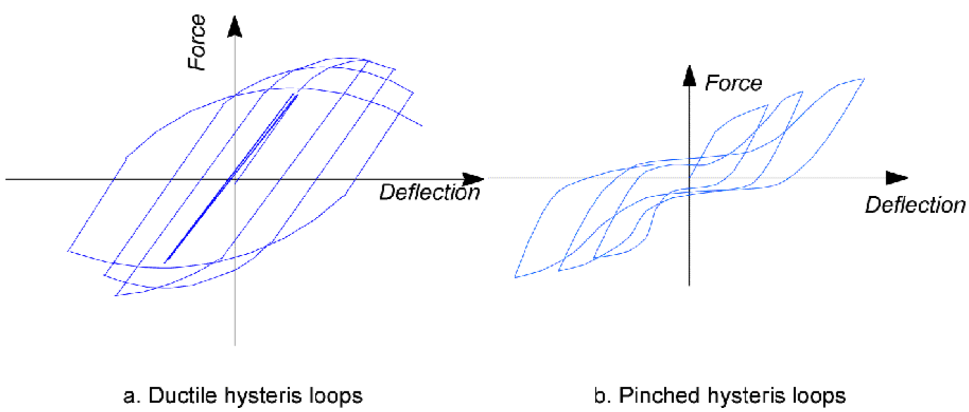

Figure C4.2-2 Typical hysteretic curves.

Some contemporary building codes, including those adopted in Canada and Europe have attempted to directly quantify the relative contribution of overstrength and inelastic behavior to the permissible reduction in design strength. Recently, the Structural Engineers Association of California proposed such an approach for incorporation into the 1997 *Uniform Building Code*. That proposal incorporated two *R* factor components, termed R_0 and R_d, to represent the reduction due to structural overstrength and inelastic behavior, respectively. The design forces are then determined by forming a composite *R*, equal to the product of the two components (see Eq. C4.2-3). A similar approach was considered for adoption into the 1997 *NEHRP Provisions*. However, this approach was not taken for several reasons. While it was acknowledged that both structural overstrength and inelastic behavior are important contributors to the *R* coefficients and that they can be quantified for individual structures, it was felt that there was insufficient research available at the current time to support implementation in the *Provisions*. In addition, there was concern that there can be significant variation between structures in the relative contribution of overstrength and inelastic behavior and that, therefore, this would prevent accurate quantification on a system-by-system basis. Finally, it was felt that this would introduce additional complexity into the *Provisions*. While it was decided not to introduce the split *R* value concept into the *Provisions* in the 1997 update cycle, this should be considered in the future as additional research on the inelastic behavior of structures becomes available and as the sophistication of design offices improves to the point that quantification of structural overstrength can be done as a routine part of the design process. As a first step in this direction, however, the factor Ω_0 was added to Table 4.3-1, to replace the previous $2R/5$ factor used for evaluation of brittle structural behavior modes in previous editions of the *Provisions*.

The *R* values, contained in the current *Provisions*, are largely based on engineering judgment of the performance of the various materials and systems in past earthquakes. 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. Since it is difficult for individual designers to judge the extent to which *R* factors should be adjusted based on the inherent redundancy of their designs, a coefficient, ρ, which is calculated based on the amount of the total lateral force resisted by any individual element, is found in *Provisions* in Sec. 4.3.3. Additional discussion of this issue is contained in that section.

In a departure from previous editions of the *Provisions*, the 1997 edition introduced an importance factor *I* into the base shear equation, which factor varies for different types of occupancies. This importance factor has the effect of adjusting the permissible response modification factor, *R*, based on the desired seismic performance for the structure. It recognizes that greater levels of inelastic behavior, correspond to increased structural damage. Thus, introducing the importance factor, *I*, allows for a reduction of the *R* value to an effective value R/I as a partial control on the amount of damage experienced by the structure under a design earthquake. Strength alone is not sufficient to obtain enhanced seismic performance. Therefore, the improved performance characteristics desired for more critical occupancies are also obtained through application of the design and detailing requirements set forth in Sec. 4.6 for each Seismic Design Category and the more stringent drift limits in Table 4.5-1. These factors, in addition to strength, are extremely important to obtaining the seismic performance desired for buildings in some Seismic Use Groups.

Sec. 4.2.1 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.

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 earlier in this section.)

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 multiple 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.

4.2.2 Combination of load effects. The load combination statements in 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. The pertinent combinations are:

$$1.2D + 1.0E + 0.5L + 0.2S \qquad \text{(Additive)}$$
$$0.9D + 1.0E \qquad \text{(Counteracting)}$$

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

The design basis expressed in Sec. 4.2.1 reflects the fact that the specified earthquake loads are at the design level without amplification by load factors; thus, for sufficiently redundant structures, a load factor of 1.0 is assigned to the earthquake load effects in Eq. 4.2-1 and 4.2-2.

4.2.2.1 Seismic load effect. In Eq. 4.2-1 and 4.2-2 , a factor of $0.2S_{DS}$ was placed on the dead load to account for the effects of vertical acceleration. The $0.2S_{DS}$ 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 ρ factor was introduced into Eq. 4.2-1 and 4.2-2 in the 1997 *Provisions*. This factor, determined in accordance with Sec. 4.3.3, relates to the redundancy inherent in the seismic-force-resisting system and is, in essence, a reliability factor, penalizing designs which are likely to be unreliable due to concentration of the structure's resistance to lateral forces in a relatively few elements.

There is very little research that speaks directly to the merits of redundancy in buildings for seismic resistance. The SAC joint venture recently studied the relationships between damage to welded steel moment frame connections and redundancy (Bonowitz et al., 1995). While this study found no specific correlation between damage and the number of bays of moment resisting framing per moment frame, it did find increased rates of damage in connections that resisted loads for larger floor areas. This study included modern low-, mid-, and high-rise steel buildings.

Another study (Wood, 1991) that addresses the potential effects of redundancy evaluated the performance of 165 Chilean concrete buildings ranging in height from 6 to 23 stories. These concrete shear wall buildings with non-ductile details and no boundary elements experienced moderately strong shaking (MMI VII to VIII) with a strong shaking duration of over 60 seconds, yet performed well. One plausible explanation for this generally good performance was the substantial amount of wall area (2 to 4 percent of the floor area) commonly used in Chile. However, Wood's study found no correlation between damage rates and higher redundancy in buildings with wall areas greater than 2 percent.

4.2.2.2 Seismic load effect with overstrength. The seismic load effect with overstrength of Sec. 4.2.2.2 is intended to address those situations where failure of an isolated, individual, brittle element can result in the loss of a complete seismic-force-resisting system or in instability and collapse. This section has evolved over several editions. In the 1991 Edition, a factor equal to $2R/5$ factor was introduced to better represent the behavior of elements sensitive to overstrength in the remainder of the seismic-force-resisting system or in other specific structural components. The particular number was selected to correlate with the $3R_w/8$ factor that had been introduced in the Structural Engineers Association of California (SEAOC) recommendations and the *Uniform Building Code*. This is a somewhat arbitrary factor that attempts to quantify the maximum force that can be delivered to sensitive elements based on historic observation that the real force that could develop in a structure may be 3 to 4 times the design levels. In the 1997 *Provisions*, an attempt was made to determine this force more rationally through the assignment of the Ω_0 factor in Table 4.3-1, dependent on the individual system. Through the use of the Ω_o coefficient, this special equation provides an estimate of the maximum forces likely to be experienced by an element.

In recent years, a number of researchers have investigated the factors that permit structures designed for reduced forces to survive design earthquakes. Although these studies have principally been focused on the development of more reliable response modification coefficients, R, they have identified the importance of structural overstrength and identified a number of sources of such overstrength. This has made it possible to replace the single $2R/5$ factor formerly contained in the *Provisions* with a more system-specific estimate, represented by the Ω_0 coefficient.

It is recognized, that no single value, whether obtained by formula related to the R factor or otherwise obtained will provide a completely accurate estimate for the overstrength of all structures with a given seismic-force-resisting system. However, most structures designed with a given seismic-force-resisting system will fall within a range of overstrength values. Since the purpose of the Ω_0 factor in Eq. 4.2-3 and 4.2-4 is to estimate the maximum force that can be delivered to a component that is sensitive to overstress, the values of this factor tabulated in Table 4.3-1 are intended to be representative of the larger values in this range for each system.

Figure C4.2-3 and the following discussion explore some of the factors that contribute to structural overstrength. The figure shows a plot of lateral structural strength vs. displacement for an elastic-perfectly-plastic structure. In addition, it shows a similar plot for a more representative real structure, that posses significantly more strength than the design strength. This real strength is represented by the lateral force F_n. Essentially, the Ω_0 coefficient is intended to be a somewhat conservative estimate of the ratio of F_n to the design strength F_E/R. As shown in the figure, there are three basic components to the overstrength. These are the design overstrength (Ω_D), the material overstrength (Ω_M) and the system overstrength (Ω_S). Each of these is discussed separately. The design overstrength (Ω_D) is the most difficult of the three to estimate. It is the difference between the lateral base shear force at which the first significant yield of the structure will occur (point 1 in the figure) and the minimum specified force given by F_E/R. To some extent, this is system dependent. Systems that are strength controlled, such as most braced frames and shear wall structures, will typically have a relatively low value of design overstrength, as most designers will seek to optimize their designs and provide a strength that is close to the minimum specified by the *Provisions*. For such structures, this portion of the overstrength coefficient could be as low as 1.0.

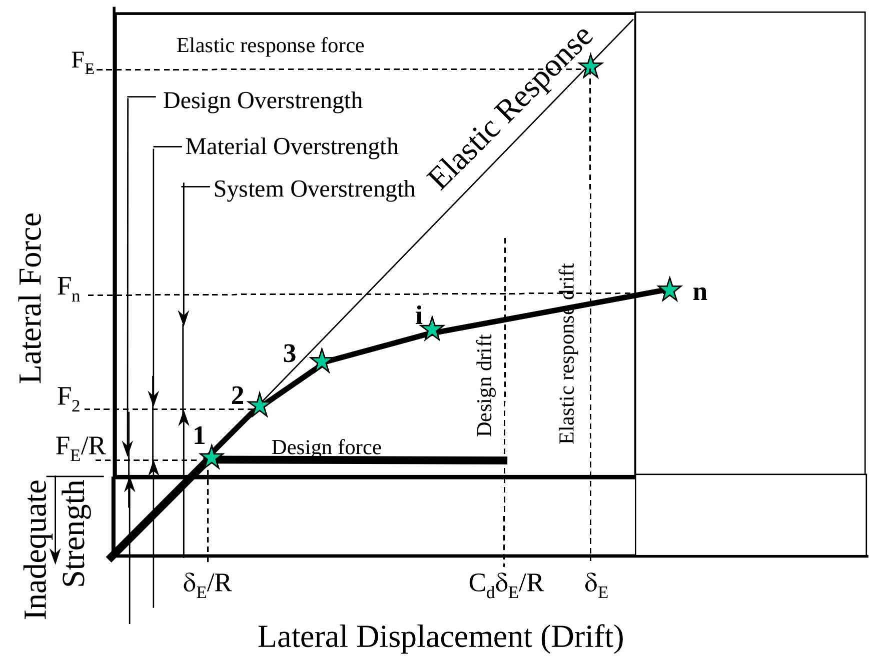

Figure C4.2-3 Factors affecting overstrength.

Drift controlled systems such as moment frames, however, will have substantially larger design overstrengths since it will be necessary to oversize the sections of such structures in order to keep the lateral drifts within prescribed limits. In a recent study of a number of special moment resisting steel frames conducted by the SAC Joint Venture design overstrengths on the order of a factor of two to three were found to exist (*Analytical Investigation of Buildings Affected by the 1994 Northridge Earthquake, Volumes 1 and 2*, SAC 95-04A and B. SAC Joint Venture, Sacramento, CA, 1995). Design overstrength also has the potential to be regionally dependent. The SAC study was conducted for frames in Seismic Design Categories D and E, which represent the most severe design conditions. For structures in Seismic Design Categories A, B and C, seismic force resistance would play a less significant role in the sizing of frame elements to control drifts, and consequently, design overstrengths for these systems would be somewhat lower. It seems reasonable to assume that this portion of the design overstrength for special moment frame structures is on the order of 2.0.

Architectural design considerations have the potential to play a significant role in design overstrength. Some architectural designs will incorporate many more and larger lateral-force-resisting elements than are required to meet the strength and drift limitations of the code. An example of this is warehouse type structures, wherein the massive perimeter walls of the structure can provide very large lateral strength. However, even in such structures, there is typically some limiting element, such as the diaphragm, that prevents the design overstrength from becoming uncontrollably large. Thus, although the warehouse structure may have very large lateral resistance in its shear walls, typically the roof diaphragm will have a lateral-force-resisting capacity comparable to that specified as a minimum by the *Provisions*.

Finally, the structural designer can affect the design overstrength. While some designers seek to optimize their structures with regard to the limitations contained in the *Provisions*, others will intentionally seek to provide greater strength and drift control than required. Typically design overstrength intentionally introduced by the designer will be on the order of 10 percent of the minimum required strength, but it may range as high as 50 to 100 percent in some cases. A factor of 1.2 should probably be presumed for this portion of the design overstrength to include the effects of both architectural and structural design overstrength. Designers who intentionally provide greater design overstrength should keep in mind that the Ω_0 factors used in their designs should be adjusted accordingly.

Material overstrength (Ω_M) results from the fact that the design values used to proportion the elements of a structure are specified by the *Provisions* to be conservative lower bound estimates of the actual probable strengths of the structural materials and their effective strengths in the as-constructed structure. It is represented in the figure by the ratio of F_2/F_1, where F_2 and F_1 are respectively the lateral force at points 2 and 1 on the curve. All structural materials have considerable variation in the strengths that can be obtained in given samples of the material from a specific grade. The design requirements typically base proportioning requirements on minimum specified values that are further reduced through strength reduction (ϕ) factors. The actual expected strength of the as-constructed structure is significantly higher than this design value and should be calculated using the mean strength of the material, based on statistical data, by removal of the ϕ factor from the design equation, and by providing an allowance for strain hardening, where significant yielding is expected to occur. Code requirements for reinforced masonry, concrete and steel have historically used a factor of 1.25 to account for the ratio of mean to specified strength and the effects of some strain hardening. Considering a typical capacity reduction factor on the order of 0.9, this would indicate that the material overstrength for systems constructed of these materials would be on the order of 1.25/0.9, or 1.4.

System overstrength (Ω_S) is the ratio of the ultimate lateral force the structure is capable of resisting, F_n in the figure, to the actual force at which first significant yield occurs, F_2 in the figure. It is dependent on the amount of redundancy contained in the structure as well as the extent to which the designer has optimized the various elements that participate in lateral force resistance. For structures, with a single lateral-force-resisting element, such as a braced frame structure with a single bay of bracing, the system overstrength (Ω_S) factor would be 1.0, because once the brace in the frame yields, the system becomes fully yielded. For structures that have a number of elements participating in lateral-force resistance, whether or not actually intended to do so, the system overstrength will be significantly larger than this, unless the designer has intentionally optimized the structure such that a complete side sway mechanism develops at the level of lateral drift at which the first actual yield occurs.

Structural optimization is most likely to occur in structures where the actual lateral-force resistance is dominated by the design of elements intended to participate as part of the lateral-force-resisting system, and where the design of those elements is dominated by seismic loads, as opposed to gravity loads. This would include concentrically braced frames and eccentrically braced frames in all Seismic Design Categories and Special Moment Frames in Seismic Design Categories D and E. For such structures, the system overstrength may be taken on the order of 1.1. For dual system structures, the system overstrength is set by the *Provisions* at an approximate minimum value of 1.25. For structures where the number of elements that actually resist lateral forces is based on other than seismic design considerations, the system overstrength may be somewhat larger. In light framed residential construction, for example, the number of walls is controlled by architectural rather than seismic design consideration. Such structures may have a system overstrength on the order of 1.5. Moment frames, the design of which is dominated by gravity load considerations can easily have a system overstrength of 2.0 or more. This effect is somewhat balanced by the fact that such frames will have a lower design overstrength related to the requirement to increase section sizes to obtain drift control. Table C4.2-1 presents some possible ranges of values for the various components of overstrength for various structural systems as well as the overall range of values that may occur for typical structures.

Table C4.2-1 Typical Range of Overstrength for Various Systems

Structural System	**Design Overstrength** Ω_D	**Material Overstrength** Ω_M	**System Overstrength** Ω_S	Ω_0
Special moment frames (steel, concrete)	1.5-2.5	1.2-1.6	1.0-1.5	2-3.5
Intermediate moment frames (steel, concrete)	1.0-2.0	1.2-1.6	1.0-2.0	2-3.5
Ordinary moment frames (steel, concrete)	1.0-1.5	1.2-1.6	1.5-2.5	2-3.5
Masonry wall frames	1.0-2.0	1.2-1.6	1.0-1.5	2-2.5
Braced frames	1.5-2.0	1.2-1.6	1.0-1.5	1.5-2
Reinforced bearing wall	1.0-1.5	1.2-1.6	1.0-1.5	1.5-2.5
Reinforced infill wall	1.0-1.5	1.2-1.6	1.0-1.5	1.5-2.5
Unreinforced bearing wall	1.0-2.0	0.8-2.0	1.0-2.0	2-3
Unreinforced infill wall	1.0-2.0	0.8-2.0	1.0-2.0	2-3
Dual system bracing and frame	1.1-1.75	1.2-1.6	1.0-1.5	1.5-2.5
Light bearing wall systems	1.0-0.5	1.2-2.0	1.0-2.0	2.5-3.5

In recognition of the fact that it is difficult to accurately estimate the amount of overstrength a structure will have, based solely on the type of seismic-force-resisting system that is present, in lieu of using the values of the overstrength coefficient Ω_0 provided in Table 4.3-1, designers are encouraged to base the maximum forces used in Eq. 4.3-3 and 4.3-4 on the results of suitable nonlinear analysis of the structure. Such analyses should use the actual expected (rather than specified) values of material and section properties. Appropriate forms of such analyses could include a plastic mechanism analysis, a static pushover analysis, or a nonlinear response history analysis. If a plastic mechanism analysis is utilized, the maximum seismic force that ever could be produced in the structure, regardless of the ground motion experienced, is estimated. If static pushover or nonlinear response history analyses are employed, the forces utilized for design as the maximum force should probably be those determined for Maximum Considered Earthquake level ground shaking demands.

While overstrength can be quite beneficial in permitting structures to resist actual seismic demands that are larger than those for which they have been specifically designed, it is not always beneficial. Some elements incorporated in structures behave in a brittle manner and can fail in an abrupt manner if substantially overloaded. The existence of structural overstrength results in a condition where such overloads are likely to occur, unless they are specifically accounted for in the design process. This is the purpose of Eq. 4.3-3 and 4.3-4.

One case where structural overstrength should specifically be considered is in the design of column elements beneath discontinuous braced frames and shear walls, such as occurs at vertical in-plane and out-of-plane irregularities. Overstrength in the braced frames and shear walls could cause buckling failure of such columns with resulting structural collapse. Columns subjected to tensile loading in which splices are made using partial penetration groove welds, a type of joint subject to brittle fracture when overloaded, are another example of a case where the seismic effect with overstrength should be used. Other design situations that warrant the use of these equations are noted throughout the *Provisions*.

Although the *Provisions* note the most common cases in which structural overstrength can lead to an undesirable failure mode, it is not possible for them to note all such conditions. Therefore, designers using the *Provisions* should be alert to conditions where the isolated independent failure of any element can lead to a condition of instability or collapse and should use the seismic effect with overstrength of Eq.

4.2-3 and 4.2-4 for the design of such elements. Other conditions which may warrant such a design approach, although not specifically noted in the *Provisions*, include the design of transfer structures beneath discontinuous lateral-force-resisting elements and the design of diaphragm force collectors to shear walls and braced frames, when these are the only method of transferring force to these elements at a diaphragm level.

4.3 SEISMIC-FORCE-RESISTING SYSTEM

4.3.1 Selection and limitations. For purposes of these seismic analyses and design requirements, building framing systems are grouped in the structural system categories shown in Table 4.3-1. These categories are similar to those contained for many years in the requirements 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.

Specification 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 4.3-1 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 dissipate 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 becomes capable of providing a nominal secondary line of resistance even though the components of the seismic-force-resisting system are designed to carry all of 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 primary 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 4.3-1 and limited by the Seismic Design Categories.

Special moment frames must meet all the design and detailing requirements of Chapter 8, 9, 10, or 11. The ductility requirements for these frame systems are appropriate for all structures anticipated to experience large inelastic demands. For this reason, they are required in zones of high seismicity with large anticipated ground shaking accelerations. In zones of lower seismicity, the inherent overstrength in typical structural designs is such that the anticipated inelastic demands are somewhat reduced, and less ductile systems may be safely employed. 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.

Requirements for composite steel-concrete systems were first introduced in the 1994 Edition. The R, Ω_0, and C_d values for the composite systems in Table 4.3-1 are similar to those for comparable systems of structural steel and reinforced concrete. The values shown in Table 4.3-1 are only allowed when the design and detailing requirements for composite structures in Chapter 10 are followed.

Inverted pendulum structures are singled out for special consideration because of their unique characteristics. These structures have little redundancy and overstrength and concentrate inelastic behavior at their bases. As a result, they have substantially less energy dissipation capacity than other systems. A number of buildings incorporating this system experienced very severe damage, and in some cases, collapse, in the 1994 Northridge earthquake.

4.3.1.1 Dual system. A dual system consists of a three-dimensional space frame made up of columns and beams that provide primary support for the gravity loads. Primary 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 Chapters 8, 9, 10, or 11. The moment frame is required to be capable of resisting at least 25 percent of the specified seismic force; this percentage is based on the judgment of the writers. 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 their 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 their 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.

4.3.1.2 Combinations of framing systems. For those cases where combinations of structural systems are employed, the designer must use judgment in selecting appropriate R, Ω_0, and C_d values. The intent of Sec. 4.3.1.2.1 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. 4.3.1.2.2 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.

4.3.1.3 - 4.3.1.6 Seismic Design Categories. General framing system requirements for the building Seismic Design Categories are given in these sections. The corresponding design and detailing requirements are given in Sec. 4.6 and Chapters 8 through 14. There are no restrictions on the selection of structural systems in Seismic Design Category A. Table 4.3-1 indicates the systems permitted in all other Seismic Design Categories.

4.3.1.4 Seismic Design Category D. Sec. 4.3.1.4 covers Seismic Design Category D, which compares roughly to California design practice for normal buildings away from major faults. In keeping with the philosophy of present codes for zones of high seismic risk, these requirements 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 requirements are arbitrary. Considerable disagreement exists regarding the adequacy of these values, and it is intended that these limitations be the subject of further study.

According to these requirements require that buildings in Category D over 160 ft (49 m) in height must 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 this chapter, 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* requirements 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. The *Provisions* require only that the true interaction behavior of the frame-shear wall (or braced frame) system be considered. If the analysis of the interacting behavior is based only on the vertical distribution of seismic lateral forces determined using the equivalent lateral force procedure of Sec. 5.2, 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 the ELF procedure.

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 only if braced frames or shear walls in any plane do not resist more than 60 percent of the seismic design force including torsional effects and the configuration of the lateral-force-resisting system is such that torsional effects result in less than a 20 percent contribution to the strength demand on the walls or frames. 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 C4.3-1 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 C4.3-2). 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 coupled shear walls or slender braced frames.

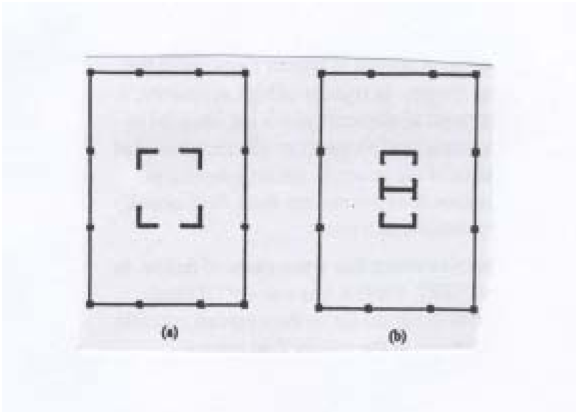

Figure C4.3-1 **Arrangement of shear walls and braced frames – not recommended. Note that the heavy lines indicate shear walls and/or braced frames.**

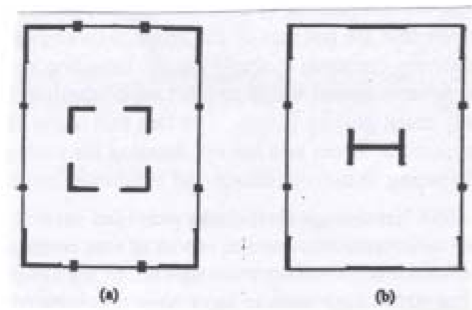

Figure C4.3-2 **Arrangement of shear walls and braced frames – recommended. Note that the heavy lines indicate shear walls and/or braced frames.**

4.3.1.4.2 Interaction effects. This section relates to the interaction of elements of the seismic-force-resisting system with elements that are not part of this system. A classic example of such interaction is the behavior of infill masonry walls used as architectural elements in a building provided with a seismic-force-resisting system composed of moment resisting frames. Although the masonry walls are not intended to resist seismic forces, at low levels of deformation they will be substantially more rigid than the moment resisting frames and will participate in lateral force resistance. A common effect of such walls is that they can create shear-critical conditions in the columns they abut by reducing the effective flexural height of these columns to the height of the openings in the walls. If these walls are neither uniformly distributed throughout the structure nor effectively isolated from participation in lateral force resistance, they can also create torsional irregularities and soft story irregularities in structures that would otherwise have regular configuration.

Infill walls are not the only elements not included in seismic-force-resisting systems that can affect a structure's seismic behavior. For example, in parking garage structures, the ramps between levels can act as effective bracing elements and resist a large portion of the seismically induced forces. They can induce large thrusts in the diaphragms where they connect, as well as large vertical forces on the adjacent columns and beams. In addition, if not symmetrically placed in the structure they can induce torsional irregularities. This section requires consideration of these potential effects.

4.3.1.6 Seismic Design Category F. Sec. 4.3.1.6 covers Category F, which is restricted to essential facilities on sites located within a few kilometers of major active faults. Because of the necessity for reducing risk (particularly in terms of providing life safety or maintaining function by minimizing damage to nonstructural building elements, contents, equipment, and utilities), the height limitations for Category F are reduced. Again, the limits—100 ft (30 m) and 160 ft (49 m)—are arbitrary and require further study. The developers of these requirements 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. 4.6 and Chapters 8 through 12.

4.3.2 Configuration. The configuration of a structure 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. There are several reasons for this poor behavior of irregular structures. In a regular structure, inelastic demands produced by strong ground shaking tend to be well distributed throughout the structure, resulting in a dispersion of energy dissipation and damage. However, in irregular structures, inelastic behavior can concentrate in the zone of irregularity, resulting in rapid failure of structural elements in these areas. In addition, some irregularities introduce unanticipated stresses into the structure which designers frequently overlook when detailing the structural system. Finally, the elastic analysis methods typically employed in the design of structures often cannot predict the distribution of earthquake demands in an irregular structure very well, leading to inadequate design in the zones of irregularity. For these reasons, these requirements are designed to encourage that buildings be designed to have regular configurations and to prohibit gross irregularity in buildings located on sites close to major active faults, where very strong ground motion and extreme inelastic demands can be experienced.

4.3.2.2 Plan irregularity. Sec. 4.3.2.2 indicates, by reference to Table 4.3-2, under what circumstances 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-force-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. In the 1997 *Provisions*, torsional irregularities were subdivided into two categories, with a category of extreme irregularity having been created. Extreme torsional irregularities are prohibited for structures located very close to major active faults and should be avoided, when possible, in all structures.

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 Sec. 5.2.4.3.

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 C4.3-3.

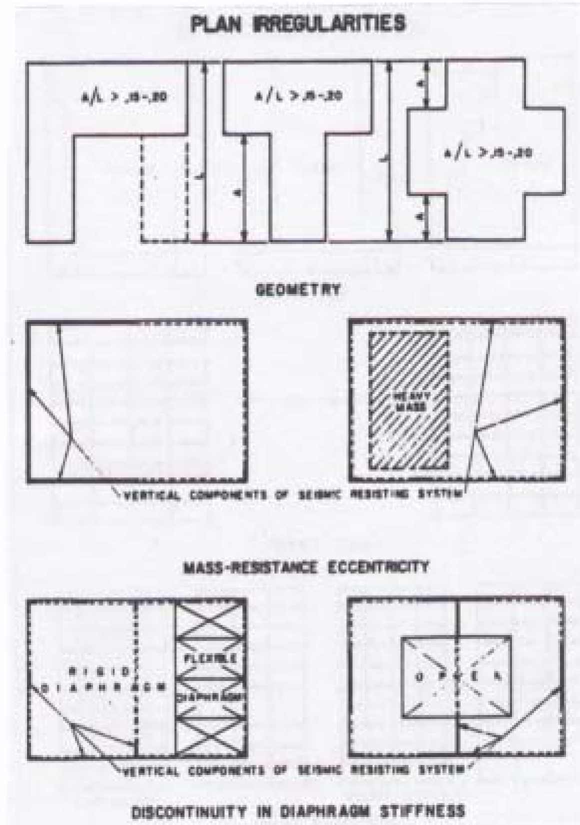

Figure 4.3-3 Building plan irregularities

Where there are discontinuities in the path of lateral force resistance, 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 about 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."

4.3.2.3 Vertical irregularity. Sec. 4.3.2.3 indicates, by reference to Table 4.3-3, under what circumstances 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. 5.2.

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 design did not compensate for the resulting decrease in stiffness that would normally occur. Examples of vertical irregularities are illustrated in Figure C4.3-4.

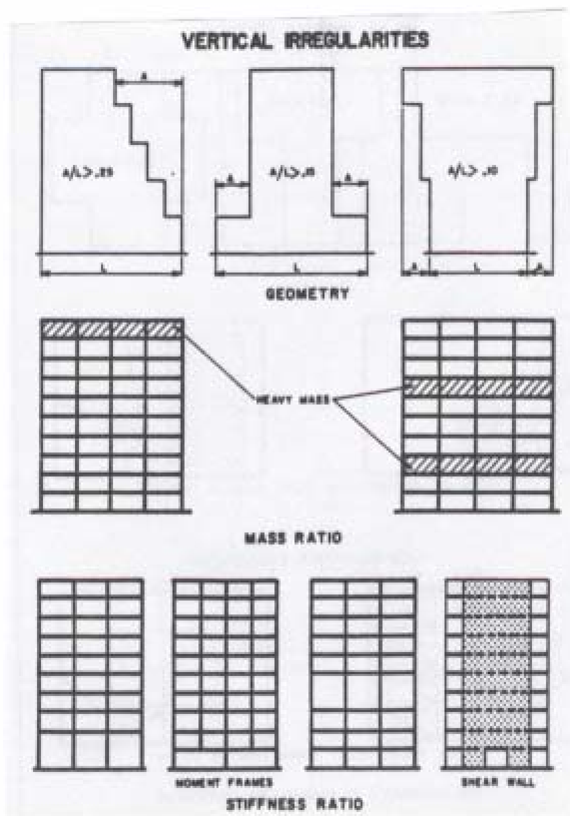

Figure C4.3-4 Building elevation irregularities

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

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

Weak story irregularities occur whenever the strength of a story to resist lateral demands is significantly less than that of the story above. This is because buildings with this configuration tend to develop all of their inelastic behavior at the weak story. This can result in a significant change in the deformation pattern of the building, with most earthquake induced displacement occurring within the weak story. This can result in extensive damage within the weak story and even instability and collapse. Note that an exception has been provided in Sec. 4.6.1.6 where there is considerable overstrength of the "weak" story.

In the 1997 *Provisions*, the soft story irregularity was subdivided into two categories with an extreme soft story category being created. Like weak stories, soft stories can lead to instability and collapse. Buildings with extreme soft stories are now prohibited on sites located very close to major active faults.

4.3.3 Redundancy. The 1997 *Provisions* introduced specific requirements intended to quantify the importance of redundancy. Many parts of the *Provisions*, particularly the response modification coefficients, *R*, were originally developed assuming that structures possess varying levels of redundancy that heretofore were undefined. *Commentary* Sec. 4.2.1 recommends that lower *R* values be used for non-redundant systems, but does not provide guidance on how to select and justify appropriate reductions. As a result, many non-redundant structures have been designed in the past using values of *R* that were intended for use in designing structures with higher levels of redundancy. For example, current *R* values for special moment resisting frames were initially established in the 1970s based on the then widespread use of complete or nearly complete frame systems in which all beam-column connections were designed to participate in the lateral-force-resisting system. High *R* values were justified by the large number of potential hinges that could form in such redundant systems, and the beneficial effects of progressive yield hinge formation described in Sec. C4.2.1. However, in recent years, economic pressures have encouraged the now prevalent use of much less redundant special moment frames with relatively few bays of moment resisting framing supporting large floor and roof areas. Similar observations have been made of other types of construction as well. Modern concrete and masonry shear wall buildings, for example, have many fewer walls than were once commonly provided in such buildings.

In order to quantify the effects of redundancy, the 1997 *Provisions* introduced the concept of a redundancy factor, ρ, that is applied to the design earthquake loads in the seismic load effect equations of Sec. 4.2.2.1, for structures in Seismic Design Categories D, E, and F. The value of the reliability factor ρ varies from 1 to 1.5. In effect this reduces the *R* values for less redundant structures and should provide greater economic incentive for the design of structures with well distributed lateral-force-resisting systems. The formulation for the equation from which ρ is derived is similar to that developed by SEAOC for inclusion in the 1997 edition of the *Uniform Building Code*. It bases the value of ρ on the floor area of the building and the parameter "r" which relates to the amount of the building's design lateral force carried by any single element.

There are many other considerations than just floor area and element/story shear ratios that should be considered in quantifying redundancy. Conceptually, element demand/capacity ratios, types of mechanisms which may form, individual characteristics of building systems and materials, building height, number of stories, irregularity, torsional resistance, chord and collector length, diaphragm spans, number of lines of resistance, and number of elements per line are all important and will intrinsically influence the level of redundancy in systems and their reliability.

The SEAOC proposed code change to the *1997 UBC* recommends addressing redundancy in irregular buildings by evaluating the ratio of element shear to design story shear, "r" only in the lower two-thirds of the height. However, in response to failures of buildings that have occurred at and above mid-heights, the writers of the *Provisions* chose to base the ρ factor on the worst "r" for the least redundant story. The resulting factor is then applied throughout the height of the building.

The Applied Technology Council, in its ATC 19 report suggests that future redundancy factors be based on reliability theory. For example, if the number of hinges in a moment frame required to achieve a minimally redundant system were established, a redundancy factor for less redundant systems could be based on the relationship of the number of hinges actually provided to those required for minimally redundant systems. ATC suggests that similar relationships could be developed for shear wall systems using reliability theory. However, much work yet remains to be completed before such approaches will be ready for adoption into the *Provisions*.

The *Provisions* limit special moment resisting frames to configurations that provide maximum ρ values of 1.25 and 1.1, respectively, in Seismic Design Categories D, and E or F, to compensate for the strength based factor in what are typically drift-controlled systems. Other seismic-force-resisting systems that are not typically drift controlled may be proportioned to exceed the maximum ρ factor of 1.5; however, it is not recommended that this be done.

4.4 STRUCTURAL ANALYSIS

4.4.1 Procedure selection. Many of the standard procedures for the analysis of forces and deformations in structures subjected to earthquake ground motion are listed below in order of increasing rigor and expected accuracy:

1. Equivalent lateral force procedure (Sec. 5.2).
2. Response spectrum (modal analysis) procedure (Sec. 5.3).
3. Linear response history procedure (Sec. 5.4).
4. Nonlinear static procedure, involving incremental application of a pattern of lateral forces and adjustment of the structural model to account for progressive yielding under load application (push-over analysis) (Appendix to Chapter 5).
5. Nonlinear response history procedure involving step-by-step integration of the coupled equations of motion (Sec. 5.5).

Each procedure becomes more rigorous if effects of soil-structure interaction are considered, either as presented in Sec. 5.6 or through a more complete analysis of this interaction, as appropriate. Every procedure improves in rigor if combined with use of results from experimental research (not described in these *Provisions*).

4.4.2 Application of loading. 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 maxima 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.

4.5 DEFORMATION REQUIREMENTS

4.5.1 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 damage to nonstructural and seismic-safety elements is a drift limit consideration.

The design story drift limits of Table 4.5-1 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 Use 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 requirements be provided for the seismic safety elements to accommodate the drift.

It should be emphasized that the drift limits, Δ_a, of Table 4.5-1 are story drifts and, therefore, are applicable to each story (that is, 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. 5.2.6.1.

Stress or strength limitations imposed by design level forces occasionally may provide adequate drift control. However, it is expected that the design of moment resisting frames, especially steel building frames, and the design of tall, narrow shear wall or braced frame buildings will be governed at least in part by drift considerations. In areas having large design spectral response accelerations, S_{DS} and S_{D1}, it is expected that seismic drift considerations will predominate for buildings of medium height. In areas having a low design spectral response accelerations and for very tall buildings in areas with large design spectral response accelerations, wind considerations generally will control, at least in the lower stories.

Due to probable first mode drift contributions, the Sec. 5.2 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 constant displacement region of a response spectra (where displacements would be essentially independent of stiffness), that the response spectrum procedure of Sec. 5.3 be used for design even when not required by Sec. 4.4.1.

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.

4.5.3 Seismic Design Categories D, E, and F. The purpose of this section is to require that the seismic-force-resisting system provide adequate deformation control to protect elements of the structure that are not part of the seismic-force-resisting system. In regions of high seismicity, it is relatively common to apply ductile detailing requirements to elements which are intended to resist seismic forces but to neglect such practices in nonstructural elements or elements intended to resist only gravity forces. The fact that many elements of the structure are not intended to resist seismic forces and are not detailed for such resistance does not prevent them from actually providing this resistance and becoming severely damaged as a result.

The 1994 Northridge earthquake provided several examples where this was a cause of failure. In a preliminary reconnaissance report of that earthquake (EERI, 1994) it was stated: "Of much significance is the observation that six of the seven partial collapses (in modern precast concrete parking structures) seem to have been precipitated by damage to the gravity load system. Possibly, the combination of large lateral deformation and vertical load caused crushing in poorly confined columns that were not detailed to be part of the lateral load resisting system." The report also noted that: "Punching shear failures were observed in some structures at slab-to-column connections such as at the Four Seasons building in Sherman Oaks. The primary lateral load resisting system was a perimeter ductile frame that performed quite well. However, the interior slab-column system was incapable of undergoing the same lateral deflections and experienced punching failures."

In response to a preponderance of evidence, SEAOC successfully submitted a change to the *Uniform Building Code* in 1994 to clarify and strengthen the existing requirements intended to require deformation compatibility. The statement in support of that code change included the following reasons: "Deformation compatibility requirements have largely been ignored by the design community. In the 1994 Northridge earthquake, deformation-induced damage to elements which were not part of the lateral-force-resisting system resulted in structural collapse. Damage to elements of the lateral-framing system, whose behavior was affected by adjoining rigid elements, was also observed. This has demonstrated a need for stronger and clearer requirements. The proposed changes attempt to emphasize the need for specific design and detailing of elements not part of the lateral system to accommodate expected seismic deformation. . . ."

Language introduced in the 1997 *Provisions* was largely based on SEAOC's successful 1995 change to the *Uniform Building Code*. Rather than implicitly relying on designers to assume appropriate levels of stiffness, the language in Sec. 4.5.3 explicitly requires that the "stiffening effects of adjoining rigid structural and nonstructural elements shall be considered and a rational value of member and restraint stiffness shall be used" for the design of components that are not part of the lateral-force-resisting system. This was intended to keep designers from neglecting the potentially adverse stiffening effects that such components can have on structures. This section also includes a requirement to address shears that can be induced in structural components that are not part of the lateral-force-resisting system since sudden shear failures have been catastrophic in past earthquakes.

The exception in Sec. 4.5.3 is intended to encourage the use of intermediate or special detailing in beams and columns that are not part of the lateral-force-resisting system. In return for better detailing, such beams and columns are permitted to be designed to resist moments and shears from unamplified deflections. This reflects observations and experimental evidence that well-detailed components can accommodate large drifts by responding inelastically without losing significant vertical load carrying capacity.

4.6 DESIGN AND DETAILING REQUIREMENTS

The design and detailing requirements for components of the seismic-force-resisting system are stated in this section. The requirements of this section are spelled out in considerable detail. 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 requirements 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 Design Categories (Tables 1.4-1 and 1.4-2), which relate to the design ground motion severities, given by the spectral response acceleration coefficients S_{DS} and S_{D1} (Chapter 3) and the Seismic Use Group (Sec. 1.2).

4.6.1 Seismic Design 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.

4.6.1.1 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 structures. Experienced seismic designers normally fill these requirements, but because some were not formally specified, they often are overlooked by inexperienced engineers.

Probably the most important single attribute of an earthquake-resistant structure is that it is tied together to act as a unit. This attribute is important not only in earthquake-resistant design, but also is indispensable in resisting high winds, floods, explosion, progressive failure, and even such ordinary hazards as foundation settlement. This section 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 $0.133S_{DS}$ (but not less than 0.05) 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. 5.2, 5.3, and 5.4 (see Sec. 4.6.3.2).

4.6.1.2 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 $400S_{DS}$ pounds per linear foot (plf) or 5,840 times S_{DS} (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.

4.6.1.3 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.

4.6.1.5 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. 5.2 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 requirements of Sec. 5.2. 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. 5.2 is based on analyses of inverted pendulums covering a wide range of practical conditions.

4.6.1.8 Collector elements. Many buildings have shear walls or other bracing elements that are not uniformly spaced around the diaphragms. Such conditions require that collector or drag members be provided. A simple illustration is shown in Figure C4.6-1.

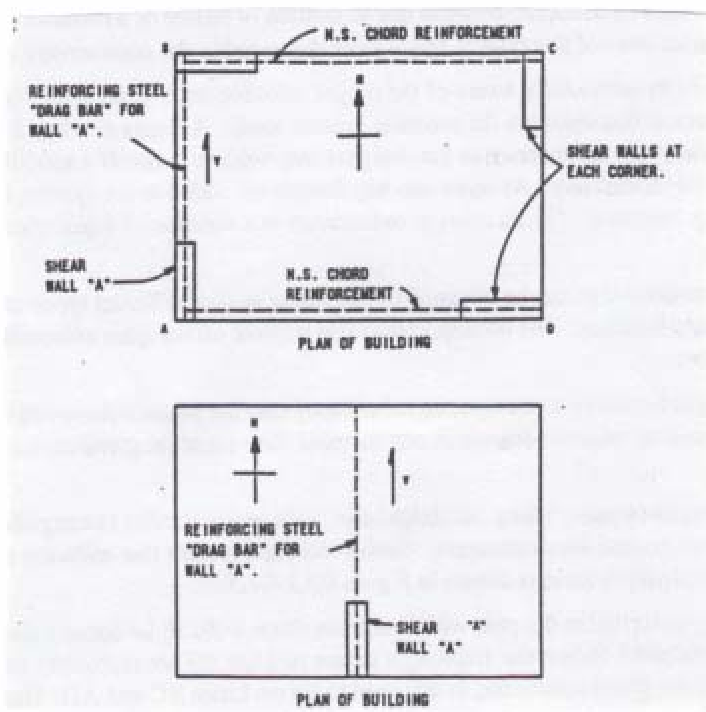

Figure 4.6-1 Collector element used to (a) transfer shears and (b) transfer drag forces from diaphragm to shear wall

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 C4.6-1 reinforcing is required to collect the shears or drag the forces from the diaphragm into the shear wall. Similar collector elements are needed for most shear walls and for some frames.

4.6.1.9 Diaphragms. Diaphragms are deep beams or trusses that distribute the lateral loads from their origin to the components where such forces are resisted. Therefore, diaphragms 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 the diaphragms are connected. The amount of deflection permitted in the diaphragm must be related to the ability of the walls to deflect (normal to the direction of force application) without failure.

A detail commonly overlooked by many engineers is the requirement to tie the diaphragm together so that it acts as a unit. Wall anchorages tend to tear off the edges of the diaphragm; thus, the ties must be extended into the diaphragm so as to develop adequate anchorage. During the San Fernando earthquake, seismic forces from the walls caused separations in roof diaphragms 20 or more feet (6 m) from the edge in several industrial buildings.

Where openings occur in shear walls or diaphragms, temperature “trim bars” alone do not provide adequate reinforcement. 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.

4.6.1.10 Anchorage of nonstructural systems. Anchorage of nonstructural systems and components of buildings is required as indicated in Chapter 6.

4.6.2 Seismic Design Category C. The material requirements in Chapters 8 through 12 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.

4.6.3 Seismic Design Categories D, E and F. 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.

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Alternative Simplified Chapter 4 Commentary

In recent years, engineers and building officials have become concerned that the *Provisions,* and the building codes based on these *Provisions*, have become increasingly complex and difficult to understand and to implement. The basic driving force for this increasing complexity is the desire of the Provisions Update Committee to provide design guidelines that will provide for the reliable performance of structures. Since the response of buildings to earthquake ground shaking is by nature, very complex, realistic accounting for these effects leads to increasingly complex provisions. However, many of the current provisions have been added as prescriptive requirements relating to the design of irregularities in structural systems. It has been recognized that in order for buildings to be reliably constructed to resist earthquakes, it is necessary that the designers have sufficient understanding of the design provisions so that they can be properly implemented. It is feared that the typical designers of smaller, simpler structures, which possibly represent more than 90 percent of construction in the United States, may have difficulty understanding what the Provisions require in their present complex form.

In recognition of this, as part of the BSSC 2000 Provisions Update Cycle, a special task force was commissioned by BSSC to develop simplified procedures, acting as an ad-hoc group reporting to TS-2. The approach was to develop a simplified set of the Provisions for easier application to low-rise, stiff structures. The procedure was designed to be used within a defined set of structures deemed to be sufficiently regular in configuration to allow a reduction of prescriptive requirements. The procedure was refined and tested over the 2000 and 2003 cycles. It is presented as a stand-alone alternate procedure to Chapter 4. Significant characteristics of this alternative chapter include the following:

1. The simplified procedure would apply to structures up to three stories high in Seismic Design Categories B, C, D, and E, but would not be allowed for systems for which the design is typically controlled by considerations of drift. The task group concluded that this approach should be limited to certain structural systems in order to avoid problems that would arise from omitting the drift check for the drift-controlled systems (steel moment frames, for example). The simplified procedure is allowed for bearing wall and building frame systems, provided that several prescriptive rules are followed that result in a torsionally resistant, regular layout of lateral-load-resisting elements.

2. Given the prescriptive rules for system configuration, the definitions, tables, and design provisions for system irregularities become unnecessary.

3. The table of basic seismic-force-resisting systems has been shortened to include only allowable systems, and deflection amplification factors are not used and have been eliminated from the table.

4. Design and detailing requirements have been consolidated into a single set of provisions that do not vary with Seismic Design Category, largely due to sections rendered unnecessary with the prohibition of system irregularities.

5. The redundancy coefficient has been removed.

6. The procedure is limited to Site Classes A to D. At the same time, it is helpful in the simplified method to have default Site Class F_a values for buildings and regions where detailed geotechnical investigations may not be available to the structural engineer. A simple definition of rock sites is provided in Sec. Alt. 4.6.1. As a practical matter, it should be known from a rudimentary geotechnical investigation whether a site is rock or soil, and so additional seismic shear wave velocity tests or special 100-ft. deep borings will not be necessary when utilizing this procedure.

The default F_a values have also been set to mitigate the tendency for the SDC to be affected by the simplified S_{DS} value.

7. Vertical shear distribution is based on tributary weight. As a result, the special formula for calculation of diaphragm forces is removed, and calculations of diaphragm forces are greatly simplified. The base shear is based on the short period plateau and does not require calculation of the period. This base value is increased 25 percent to account for the vertical distribution method as well as other simplifications. A calibration study, Figure CAlt.4-1, covering a wide range of conditions indicates that the 25 percent adequately covers the simplifications without being overly conservative.

8. Simple rigidity analysis will be required for rigid diaphragm systems, but analysis of accidental torsion and dynamic amplification of torsion would not be required. Untopped metal deck, wood panel, or plywood sheathed diaphragms may be considered flexible, representing another simplification in calculations.

9. Calculations for period, drift, or P-delta effects need not be performed. 1percent drift is assumed when needed by requirements not covered in the simplified provisions. For example, in ACI 318, gravity columns are required to be designed for the calculated drift or to be specially detailed.

Calibration Study		**Simplified Lateral Force Analysis Procedure**				**V =1.25S_{DS} / R**
Fa =		1 for rock		1.4 for soil		
Sds =		0.67		0.93		
1.25*Sds* =		0.83		1.17		

***Fa* Values**		*Z*<=0.067	*Z*=0.13	*Z*=0.20	*Z*=0.27	*Z*>=0.33
Site Class		*Ss*<=0.25	*Ss*=0.50	*Ss*=0.75	*Ss*=1.00	*Ss*>=1.25
A	(hard rock)	***0.80***	***0.80***	0.80	**0.80**	**0.80**
B	(rock)	***1.00***	1.00	**1.00**	**1.00**	**1.00**
C	(soft rock)	***1.20***	1.20	**1.10**	**1.00**	**1.00**
D	(stiff soil)	***1.60***	1.40	**1.20**	**1.10**	**1.00**

Ratio of (Simplified S_{DS}) / (S_{DS} = Fa x 2/3 x S_S)

Site Class		*Ss*<=0.25	*Ss*=0.50	*Ss*=0.75	*Ss*=1.00	*Ss*>=1.25
A	(hard rock)	***1.25***	***1.25***	1.25	**1.25**	**1.25**
B	(rock)	***1.00***	1.00	**1.00**	**1.00**	**1.00**
C	(soft rock)	***0.83***	0.83	**0.91**	**1.00**	**1.00**
D	(stiff soil)	***0.88***	1.00	**1.17**	**1.27**	**1.40**

Ratio of base shear for all buildings and overturning moment for one-story buildings

Site Class		*Ss*<=0.25	*Ss*=0.50	*Ss*=0.75	*Ss*=1.00	*Ss*>=1.25
A	(hard rock)	***1.56***	***1.56***	1.56	**1.56**	**1.56**
B	(rock)	***1.25***	1.25	**1.25**	**1.25**	**1.25**
C	(soft rock)	***1.04***	1.04	**1.14**	**1.25**	**1.25**
D	(stiff soil)	***1.09***	1.25	**1.46**	**1.59**	**1.75**

Average net conservatism in overturning moment for two-story buildings

Equal floor masses and first story height equal or up to 1.5 x second story

Site Class		*Ss*.<=0.25	*Ss*=0.50	*Ss*=0.75	*Ss*=1.00	*Ss*>=1.25
A	(hard rock)	***1.44***	***1.44***	1.44	**1.44**	**1.44**
B	(rock)	***1.15***	1.15	**1.15**	**1.15**	**1.15**
C	(soft rock)	***0.96***	0.96	**1.05**	**1.15**	**1.15**
D	(stiff soil)	***1.01***	1.15	**1.34**	**1.46**	**1.61**

Average net conservatism in overturning moment for three-story buildings

Equal floor masses and first story height equal or up to 1.5 x typical story

Site Class		*Ss*<=0.25	*Ss*=0.50	*Ss*=0.75	*Ss*=1.00	*Ss*>=1.25
A	(hard rock)	***1.38***	***1.38***	1.38	**1.38**	**1.38**
B	(rock)	***1.10***	1.10	**1.10**	**1.10**	**1.10**
C	(soft rock)	***0.92***	0.92	**1.00**	**1.10**	**1.10**
D	(stiff soil)	***0.96***	1.10	**1.28**	**1.40**	**1.54**

Bold values indicates Seismic Design Category D in the Equivalent Lateral Force Procedure.
Bold Italic values indicates Seismic Design Category B in the Equivalent Lateral Force Procedure.

Figure CAlt.4-1 Calibration Study.

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Chapter 5 Commentary

STRUCTURAL ANALYSIS PROCEDURES

5.1 GENERAL

The equivalent lateral force (ELF) procedure specified in Sec. 5.2 is similar in its basic concept to SEAOC recommendations in 1968, 1973, and 1974, but several improved features have been incorporated. A significant revision to this procedure, which more closely reflects the ground motion response spectra, was adopted in the 1997 *Provisions* in parallel with a similar concept developed by SEAOC.

The modal superposition method 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, for a number of years; however, its use is also becoming more common for ordinary structures as well. Prior to the 1997 edition of the *Provisions*, the modal analysis procedure specified in Sec. 5.3 was simplified from the general case by restricting consideration to lateral motion in a single plane. Only one degree of freedom was required per floor for this type of analysis. In recent years, with the advent of high-speed, desktop computers, and the proliferation of relatively inexpensive, user-friendly structural analysis software capable of performing three dimensional modal analyses, such simplifications have become unnecessary. Consequently, the 1997 *Provisions* adopted the more general approach describing a three-dimensional modal analysis of the structure. When modal analysis is specified by the *Provisions*, a three-dimensional analysis generally is required except in the case of highly regular structures or structures with flexible diaphragms.

The ELF procedure of Sec. 5.2 and the response spectrum procedure of Sec. 5.3 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 mass and stiffness distribution. In the ELF procedure, the distribution is based on simplified formulas that are appropriate for regular structures as specified in Sec. 5.2.3. Otherwise, the two procedures are subject to the same limitations.

The simplifications inherent in the ELF procedure result in approximations that are 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. 4.3.2.2) 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. The modal analysis method introduced in the 1997 *Provisions* includes a general model that is more appropriate for the analysis of such structures. It requires at least three degrees of freedom per floor—two for translational motion and one for torsional motion.

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 translational and three rotational) connected by generalized stiffness elements.

The ELF procedure (Sec. 5.2) and the response spectrum procedure 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. The nonlinear static (or so-called pushover) procedure is a method to more accurately account for irregular strength distribution. However, it also has limitations and is not particularly applicable to tall structures or structures with relatively long fundamental periods of vibration.

The actual strength properties of the various components of a structure 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 earthquake research studies of inelastic structural response. 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, for motion in the direction along which the structure is being analyzed; otherwise at least three degrees of freedom per floor, two translational and one torsional, should be included. It should be recognized that the results of a nonlinear response history analysis of such mathematical structural models are only as good as are the models chosen to represent the structure 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- and three-dimensional inelastic analyses of reasonably simple 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 analysis, there is no one method that can be applied to all types of structures. Further, the reliability of the analytical results are sensitive to:

1. The number and appropriateness of the input motion records,
2. The practical limitations of mathematical modeling including interacting effects of inelastic elements,
3. The nonlinear solution algorithms, and
4. The assumed hysteretic behavior of members.

Because of these sensitivities and limitations, the maximum base shear produced in an inelastic analysis should not be less than that required by Sec. 5.2.

The least rigorous analytical procedure that may be used in determining the design seismic forces and deformations in structures depends on the Seismic Design Category and the structural characteristics (in particular, regularity). Regularity is discussed in Sec. 4.3.2.

Except for structures assigned to Seismic Design Category A, the ELF procedure is the minimum level of analysis except that a more rigorous procedure is required for some Category D, E and F structures as identified in Table 4.4-1. The modal analysis procedure adequately addresses vertical irregularities of stiffness, mass, or geometry, as limited by the *Provisions*. Other irregularities must be carefully considered.

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

1. Structures with irregular mass and stiffness properties in which case the simple equations for vertical distribution of lateral forces (Eq. 5.2-10 and 5.2-11) may lead to erroneous results;
2. Structures (regular or irregular) in which the lateral motions in two orthogonal directions and the torsional motion are strongly coupled; and

3. Structures 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.

Structures with certain types of vertical irregularities may be analyzed as regular structures in accordance with the requirements of Sec. 5.2. These structures are generally referred to as setback structures. 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 structures, 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. 5.3.
2. The base and tower portions may be analyzed as separate structures in accordance with the following:
   a. The tower may be analyzed in accordance with the procedures in Sec. 5.2 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. 5.2 using the height of the base portion of h_n and with the gravity load and seismic base shear forces of the tower portion acting at the top level of the base portion.

The design requirements in Sec. 5.3 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 structures 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 structures, the following procedure should be used to determine whether the modal analysis procedures of Sec. 5.3 should be used:

1. Compute the story shears using the ELF procedure specified in Sec. 5.2.
2. On this basis, approximately dimension the structural members, and then compute the lateral displacements of the floor.
3. Replace h in Eq. 5.2-11 with these displacements, and recompute the lateral forces to obtain the revised story shears.
4. If at any story the recomputed story shear differs from the corresponding value as obtained from the procedures of Sec. 5.2 by more than 30 percent, the building should be analyzed using the procedure of Sec. 5.3. 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. 5.3 are not required.

Application of this procedure to these structures requires far less computational effort than the use of the response spectrum procedure of Sec. 5.3. In the majority of the structures, 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.

5.2 EQUIVALENT LATERAL FORCE PROCEDURE

This section discusses the equivalent lateral force (ELF) procedure for seismic analysis of structures.

5.2.1 Seismic base shear. The heart of the ELF procedure is Eq. 5.2-1 for base shear, which gives the total seismic design force, V, in terms of two factors: a seismic response coefficient, C_s, and the seismic weight, W. The seismic response coefficient C_s, is obtained from Eq. 5.2-2 and 5.2-3 based on the design spectral response acceleration parameters, S_{DS} and S_{D1}. These acceleration parameters and the derivation of the response spectrum is discussed more fully in the *Commentary* for Chapter 3. The seismic weight is discussed in *Commentary* Sec. 1.5.1.

The base shear formula and the various factors contained therein were arrived at as explained below.

Elastic acceleration response spectrum. See the *Commentary* to Chapter 4 for a full discussion of the shape of the spectrum accounting for dynamic response amplification and the effect of site response.

Elastic design spectrum. The elastic acceleration response spectrum for earthquake motions has a descending branch for longer values of T, the period of vibration of the system, that varies roughly as a function of $1/T$. In previous editions of the *Provisions*, the actual response spectra that varied in a 1/T relationship were replaced with design spectra that varied in a $1/T^{2/3}$ relationship. This was intentionally done to provide added conservatism in the design of tall structures, as well as to account for the effects of higher mode participation. In the development of the 1997 *Provisions*, a special task force, known as the Seismic Design Procedures Group (SDPG), was convened to develop a method for using new seismic hazard maps, developed by the USGS in the *Provisions*. Whereas older seismic hazard maps provided an effective peak ground acceleration coefficient, C_a, and an effective peak velocity-related acceleration coefficient, C_v, the new maps directly provide parameters that correspond to points on the response spectrum. It was the recommendation of the SDPG that the true shape of the response spectrum, represented by a 1/T relationship, be used in the base shear equation. In order to maintain the added conservatism for tall and high occupancy structures, formerly provided by the design spectra which utilized a $1/T^{2/3}$ relationship, the 1997 *Provisions* adopted an occupancy importance factor I into the base shear equation. This I factor, which has a value of 1.25 for Seismic Use Group II structures and 1.5 for Seismic Use Group III structures has the effect of raising the design spectrum for taller, high occupancy structures, to levels comparable to those for which they were designed in previous editions of the *Provisions*.

Although the introduction of an occupancy importance factor in the 1997 edition adjusted the base shear to more conservative values for large buildings with higher occupancies, it did not address the issue of accounting for higher mode effects, which can be significant in longer period structures—those with fundamental modes of vibration significantly larger than the period T_s, at which the response spectrum changes from one of constant response acceleration (Eq. 5.2-2) to one of constant response velocity (Eq. 5.2-3).

Equation 5.2-3 could be modified to produce an estimate of base shear that is more consistent with the results predicted by elastic response spectrum methods. Some suggestions for such modifications may be found in Chopra (1995). However, it is important to note that even if the base shear equation were to simulate results of an elastic response spectrum analysis more accurately, most structures respond to design level ground shaking in an inelastic manner. This inelastic response results in different demands than are predicted by elastic analysis, regardless of how "exact" the analysis is. Inelastic response behavior in multistory buildings could be partially accounted for by other modifications to the seismic coefficient C_s. Specifically, the coefficient could be made larger to limit the ductility demand in multistory buildings to the same value as for single-degree-of-freedom systems. Results supporting such an approach may be found in (Chopra, 1995) and in (Nassar and Krawinkler, 1991).

The above notwithstanding, the equivalent lateral force procedure is intended to provide a relatively straightforward design approach where complex analyses, accurately accounting for dynamic and inelastic response effects, are not warranted. Rather than making the procedure more complex, so that it would be more appropriate for structures with significant higher mode response, in the 2000 edition of the *Provisions* application of this technique to structures assigned to Seismic Design Categories D, E, and F is limited to those where higher mode effects are not significant. Given the widespread use of computer-assisted analysis for major structures, it was felt that these limitations on the application of the equivalent lateral force procedure would not be burdensome. It should be noted that particularly for tall structures, the use of dynamic analysis methods will not only result in a more realistic characterization of the distribution of inertial forces in the structure, but may also result in reduced forces, particularly with regard to overturning demands. Therefore, use of a dynamic analysis method is recommended for such structures, regardless of the Seismic Design Category.

Historically, the ELF analytical approach has been limited in application in Seismic Design Categories D, E, and F to regular structures with heights of 240 ft (70 m) or less and irregular structures with heights of 100 ft (30 m) or less. Following recognition that the use of a base shear equation with a $1/T$ relationship underestimated the response of structures with significant higher mode participation, a change in the height limit for regular structures to 100 ft (30 m) was contemplated. However, the importance of higher mode participation in structural response is a function both of the structure's dynamic properties, which are dependent on height, mass and the stiffness of various lateral force resisting elements, and of the frequency content of the ground shaking, as represented by the response spectrum. Therefore, rather than continuing to use building height as the primary parameter used to control analysis procedures, it was decided to limit the application of the ELF to those structures in Seismic Design Categories D, E, and F having fundamental periods of response less than 3.5 times the period at which the response spectrum transitions from constant response acceleration to constant response velocity. This limit was selected based on comparisons of the base shear calculated by the ELF equations to that predicted by response spectrum analysis for structures of various periods on five different sites, representative of typical conditions in the eastern and western United States. For all 5 sites, it was determined that the ELF equations conservatively bound the results of a response spectrum analysis for structures having periods lower than the indicated amount.

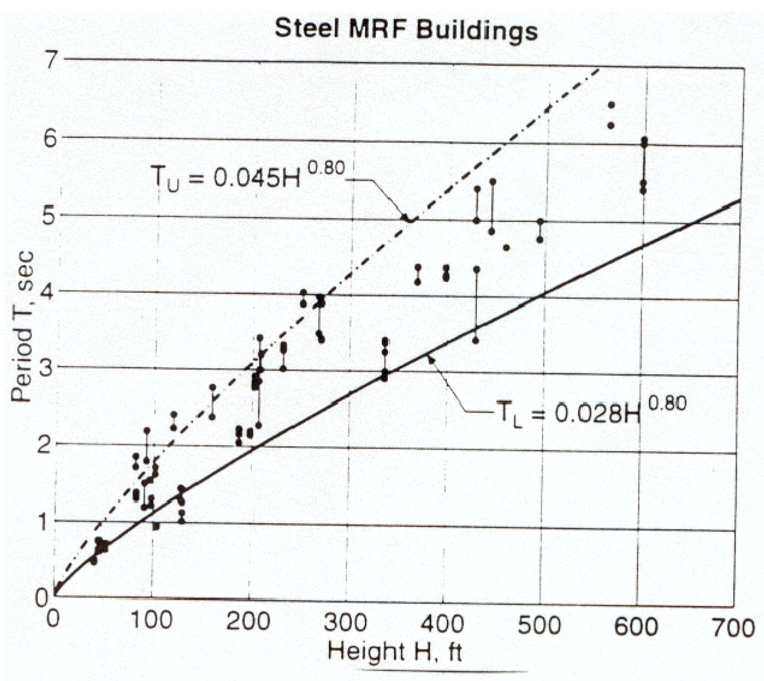

Figure C5.2-2 Measured building period for moment-resisting steel frame structures.

Response modification factor. The factor *R* in the denominator of Eq. 5.2-2 and 5.2-3 is an empirical response reduction factor intended to account for damping, overstrength, 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 (that is, 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 4.3-1 in the *Provisions* stipulates *R* factors for different types of building systems using several different structural materials. The coefficient *R* ranges in value from a minimum of 1¼ 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 4.3-1 is explained in the *Commentary* to Sec. 4.2.1.

The effective value of *R* used in the base shear equation is adjusted by the occupancy importance factor *I*. The value of *I*, which ranges from 1 to 1.5, has the effect of reducing the amount of ductility the structure will be called on to provide at a given level of ground shaking. However, it must be recognized that added strength, by itself, is not adequate to provide for superior seismic performance in buildings with critical occupancies. Good connections and construction details, quality assurance procedures, and limitations on building deformation or drift are also important to significantly improve the capability for maintenance of function and safety in critical facilities and those with a high-density occupancy. Consequently, the reduction in the damage potential of critical facilities (Group III) is also handled by using more conservative drift controls (Sec. 4.5.1) and by providing special design and detailing requirements (Sec. 4.6) and materials limitations (Chapters 8 through 12).

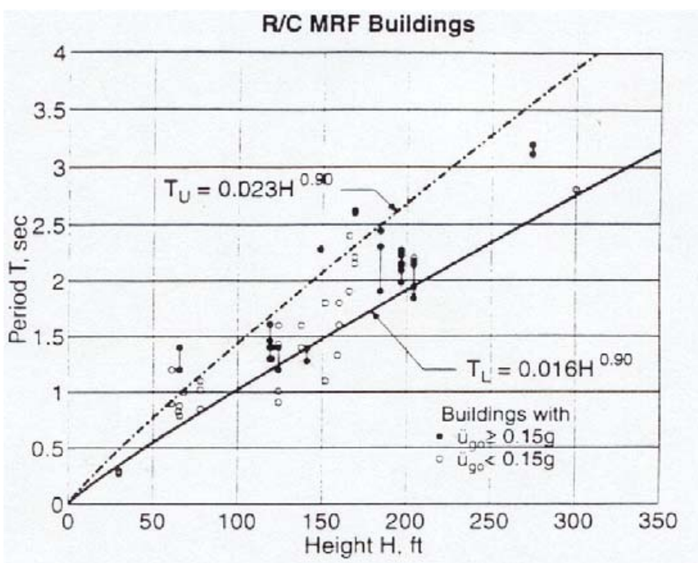

Figure C5.2-1 Measured building period for reinforced concrete frame structures.

5.2.2 Period determination. In the denominator of Eq. 5.2-3, *T* is the fundamental period of vibration of the structure. It is preferable that this be determined using modal analysis methods and the principles of structural mechanics. However, methods of structural mechanics cannot be employed to calculate the vibration period before a structure has been designed. Consequently, this section provides an approximate method that can be used to estimate the period, with minimal information available on the design. It is based on the use of simple formulas that involve only a general description of the type of structure (such as steel moment frame, concrete moment frame, shear wall system, braced frame) and overall dimensions (such as height and plan length) to estimate the period of vibration in order to calculate an initial base shear and proceed with a preliminary design.

It is advisable that this base shear and the corresponding value of T be conservative.

Even for final design, use of an unrealistically large value for T is unconservative. Thus, the value of T used in design should be smaller than the period calculated for the bare frame of the building. Equations 5.2-6, 5.2-7, and 5.2-8 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 the value of T calculated using more exact methods, based on T_a and the factor C_u. The coefficient C_u is intended to reflect the likelihood 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. Where the design of a structure is actually "controlled" by wind, the use 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 a function of $1/T$ and assuming that the lateral forces are distributed linearly over the height and that the deflections are controlled by drift limitations, a simple calculation of the period of vibration by Rayleigh's method 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. Based on this, for many years Eq. 5.2-6 appeared in the *Provisions* in the form:

$$T_a = C_t h_n^{3/4}$$

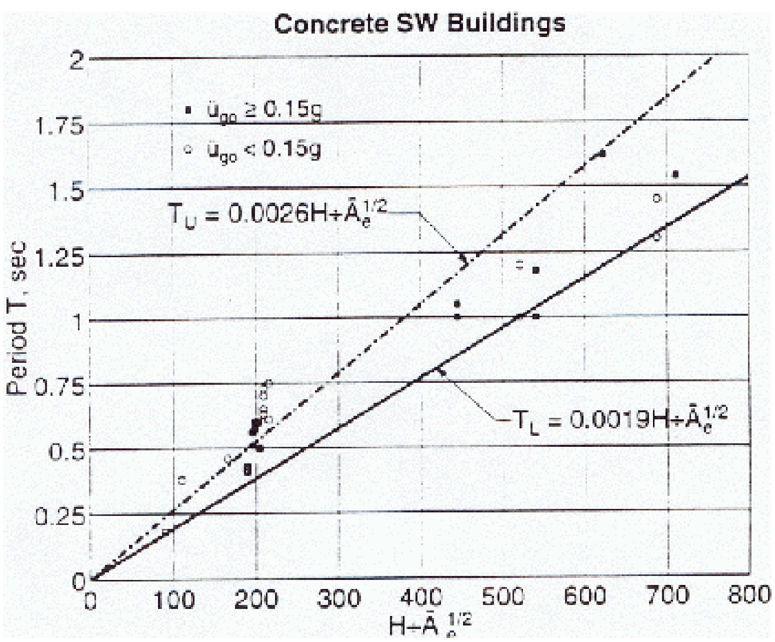

Figure C5.2-3 Measured building period for concrete shear wall structures.

A large number of strong motion instruments have been placed in buildings located within zones of high seismic activity by the U.S. Geological Survey and the California Division of Mines and Geology. Over the past several years, this has allowed the response to strong ground shaking for a significant number of these buildings to be recorded and the fundamental period of vibration of the buildings to be calculated.

Figures C5.2-1, C5.2-2, and C5.2-3, respectively, show plots of these data as a function of building height for three classes of structures. Figure C5.2-1 shows the data for moment-resisting concrete frame buildings; Figure C5.2-2, for moment-resisting steel frame buildings; and Figure C5.2-3, for concrete shear wall buildings. Also shown in these figures are equations for lines that envelop the data within approximately a standard deviation above and below the mean.

For the 2000 *Provisions*, Eq. 5.2-6 is revised into a more general form allowing the statistical fits of the data shown in the figures to be used directly. The values of the coefficient C_r and the exponent x given in Table 5.2-2 for these moment-resisting frame structures represent the lower bound (mean minus one standard deviation) fits to the data shown in Figures C5.2-1 and C5.2-2, respectively, for steel and concrete moment frames. Although updated data were available for concrete shear wall structures, these data do not fit well with an equation of the form of Eq. 5.2-6. This is because the period of shear wall buildings is highly dependent not only on the height of the structure but also on the amount of shear wall present in the building. Analytical evaluations performed by Chopra and Goel (1997 and 1998) indicate that equations of the form of Eq. 5.2-8 and 5.2-9 provide a reasonably good fit to the data. However, the form of these equations is somewhat complex. Therefore, the simpler form of Eq. 5.2-6 contained in earlier editions of the *Provisions* was retained with the newer, more accurate formulation presented as an alternative.

Updated data for other classes of construction were not available. As a result, the C_r and x values for other types of construction shown in Table 5.2-2 are values largely based on limited data obtained from the 1971 San Fernando earthquake that have been used in past editions of the *Provisions*. The optional use of $T = 0.1N$ (Eq. 5.2-7) is an approximation for low to moderate height frames that has long been in use.

In earlier editions of the *Provisions*, the C_u coefficient varied from a value of 1.2 in zones of high seismicity to a value of 1.7 in zones of low seismicity. The data presented in Figures C5.2-1, C5.2-2, and C5.2-3 permit direct evaluation of the upper bound on period as a function of the lower bound, given by Eq. 5.2-6. This data indicates that in zones of high seismicity, the ratio of the upper to lower bound may more properly be taken as a value of about 1.4. Therefore, in the 2000 *Provisions*, the values in Table 5.2-1 were revised to reflect this data in zones of high seismicity while retaining the somewhat subjective values contained in earlier editions for the zones of lower seismicity.

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. 5.2-6. 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.

Although the approximate methods of Sec. 5.2.2.1 can be used to determine a period for the design of structures, the fundamental period of vibration of the seismic-force-resisting system should be calculated according to established methods of mechanics. 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:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} F_i \delta_i}} \tag{C5.2-1}$$

where:

F_i = the seismic lateral force at Level i,

w_i = the seismic weight 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 due to 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 participate in the behavior of the structure even though the designer may not rely on them to contribute 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_uT_a is imposed as a safeguard.

5.2.3 Vertical distribution of seismic forces. The distribution of lateral forces over the height of a structure 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 structure, and the shapes of vibration modes that, in turn, depend on the distribution of mass and stiffness over the height. The basis of this method is discussed below. In structures having only minor irregularity of mass or stiffness over the height, the accuracy of the lateral force distribution as given by Eq. 5.2-11 is much improved by the procedure described in the last portion of Sec. 5.1 of this commentary. The lateral force at each level, x, due to response in the first (fundamental) natural mode of vibration is given by Eq. C5.2-2 as follows:

$$f_{x1} = V_1 \left[\frac{w_x \phi_{x1}}{\sum_{i=1}^{n} w_1 \phi_{i1}} \right] \tag{C5.2-2}$$

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. 5.3-7 in Sec. 5.3.5 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. 5.2-11 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 structures and that, in regular structures, the fundamental vibration mode departs little from a straight line. This, along with the matters discussed above, provides the basis for Eq. 5.2-11 with $k = 1$ for structures having a fundamental vibration period of 0.5 seconds or less.

It has been demonstrated that although the earthquake response of long period structures is primarily due to the fundamental natural mode of vibration, the influence of higher modes of vibration can be significant and, in regular structures, the fundamental vibration mode lies approximately between a straight line and a parabola with the vertex at the base. Thus, Eq. 5.2-11 with $k = 2$ is appropriate for structures 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.

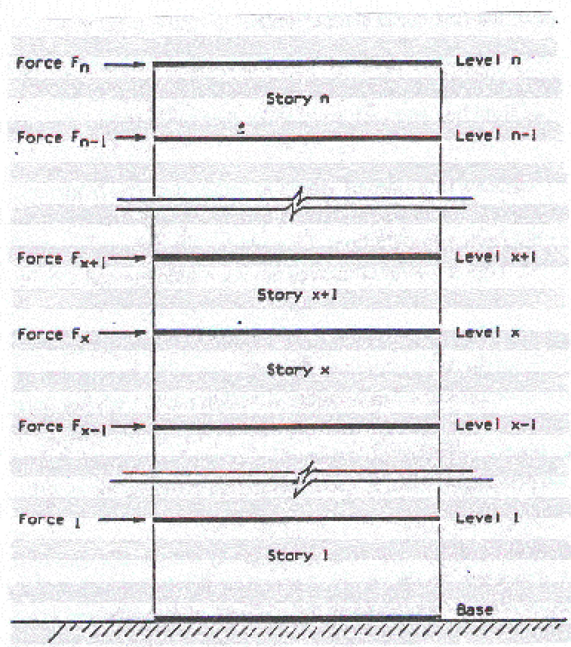

Figure C5.2-4 Description of story and level.
The shear at Story *x* (V_x) is the sum of all the lateral forces at and above Story *x* (F_x through F_n).

5.2.4 Horizontal shear distribution. The story shear in any story is the sum of the lateral forces acting at all levels above that story. Story *x* is the story immediately below Level *x* (Figure C5.2-4). Reasonable and consistent assumptions regarding the stiffness of concrete and masonry elements may be used for analysis in distributing the shear force to such elements connected by a horizontal diaphragm. Similarly, the stiffness of moment or braced frames will establish the distribution of the story shear to the vertical resisting elements in that story.

5.2.4.1 and 5.2.4.2 Inherent and accidental 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, which is 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,” which is computed as the story shear times the “accidental eccentricity,” equal to 5 percent of the dimension of the structure (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. 5.2.4.2 which implies that the dimension of the structure 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 (for example, first, all of them to the left and, then, to the right).

Dynamic analyses assuming linear behavior indicate that the torsional moment due to eccentricity between centers of mass and resistance may significantly exceed M_t (Newmark and Rosenblueth, 1971). However, such dynamic magnification is not included in the *Provisions*, partly because its significance is not well understood for structures 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 structure, 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 no 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.

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 nearly 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 structure 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 could be based on 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 and the elements perpendicular to the direction of motion are ignored, the story shear can be distributed to the vertical components based on their tributary areas.

5.2.4.3 Dynamic amplification of torsion. There are indications that the 5 percent accidental eccentricity may be too small in some structures since they may develop torsional dynamic instability. Some examples are the upper stories of tall structures having little or no nominal eccentricity, those structures where the calculations of relative stiffnesses of various elements are particularly uncertain (such as 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 (for example, some prestressed concrete frames). The amplification factor for torsionally irregular structures (Eq. 5.2-13) was introduced in the 1988 Edition as an attempt to account for some of these problems in a controlled and rational way.

5.2.5 Overturning. This section requires that the structure be designed to resist overturning moments statically consistent with the design story shears. In the 1997 and earlier editions of the *Provisions*, the overturning moment was modified by a factor, τ, to account, in an approximate manner, for the effects of higher mode response in taller structures. In the 2000 edition of the *Provisions*, the equivalent lateral force procedure was limited in application in Seismic Design Categories D, E, and F to structures that do not have significant higher mode participation. As a result it was possible to simplify the design procedure by eliminating the τ factor. Under this new approach tall structures in Seismic Design Categories B and C designed using the equivalent lateral force procedure will be designed for somewhat larger overturning demands than under past editions of the *Provisions*. This conservatism was accepted as an inducement for designers of such structures to use a more appropriate dynamic analysis procedure.

In the design of the foundation, the overturning moment calculated at the foundation-soil interface may be reduced to 75 percent of the calculated value using Eq. 5.2-14. 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.

5.2.6 Drift determination and *P*-delta effects. This section defines the design story drift as the difference of the deflections, δ_x, at the top and bottom of the story under consideration. The deflections, δ_x, are determined by multiplying the deflections, δ_{xe} (determined from an elastic analysis), by the deflection amplification factor, C_d, given in Table 4.3-1. The elastic analysis is to be made for the seismic-force-resisting system using the prescribed seismic design forces and considering the structure 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. 5.2.3. For frame structures, the axial deformations from bending effects, although contributing to the overall structural 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. 4.5.1, 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 structure, T (calculated without the limit $T \leq C_u T_a$ specified in Sec. 5.2.2), 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. 5.2.6.2 are significant, the design story drift must be increased by the resulting incremental factor.

The *P*-delta effects in a given story are due to the eccentricity of the gravity load above that story. If the story drift due to the lateral forces prescribed in Sec. 5.2.3 were Δ, the bending moments in the story would be augmented by an amount equal to Δ times the gravity load above the story. The ratio of the *P*-delta moment to the lateral force story moment is designated as a stability coefficient, θ, in Eq. 5.2-16. 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 structure 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, 1985). 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 structures, the *P*-delta requirements remained as originally written until revised for the 1991 Edition.

There was increasing evidence that the use of elastic 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. 5.2-16 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 is 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. This example illustrates that there could be an extreme impact on the requirements if a change were 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 C4.2-3). This overstrength likely protects structures from stability-related failures.

2. The likelihood of a failure due to instability decreases with increased intensity of expected ground-shaking. This is due to the fact that the stiffness of most structures designed for extreme ground motion is significantly greater than the stiffness of the same structure 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, recent editions of the *Provisions* regarding *P*-delta amplifiers have remained from the 1991 Editions.

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 8 through 14 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*, taking β equal to 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, for code-required wind resistance, or simply a feature of other aspects of the design. 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.

5.3 RESPONSE SPECTRUM PROCEDURE

Modal analysis (Newmark and Rosenblueth, 1971; Clough and Penzien, 1975; Thomson, 1965; Wiegel, 1970) 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 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 ELF procedure of Sec. 5.2 is simply a first mode application of this technique, which assumes all of the structure’s mass is active in the first mode. The purpose of modal analysis is to obtain the maximum response of the structure in each of its important modes, which are then summed in an appropriate manner. 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 traditionally used in building codes (the SEAOC recommendations and the *UBC*) and the ELF procedure in Sec. 5.2. Thus, the coefficient C_{sm} in Eq. 5.3-3 and the distribution equations, Eq. 5.3-1 and 5.3-2, are the counterparts of Eq. 5.2-10 and 5.2-11. This correspondence helps clarify the fact that the simplified modal analysis contained in Sec. 5.3 is simply an attempt to specify the equivalent lateral forces on a structure in a way that directly reflects the individual dynamic characteristics of the structure. 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. 5.2.

5.3.2 Modes. This section defines the number of modes to be used in the analysis. For many structures, including low-rise structures 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 structure. For high-rise structures, however, more than three modes may be required to adequately determine the forces for design. This section provides a simple rule that the combined participating mass of all modes considered in the analysis should be equal to or greater than 90 percent of the effective total mass in each of two orthogonal horizontal directions.

5.3.3 Modal properties. 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} is Sec. 5.3.4. Because the periods of the modes contemplated in these requirements are those associated with moderately large, but still essentially linear, structural 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 structure when completed or the response to small earthquake motions because of the stiffening effects of nonstructural and architectural components of the structure at small amplitudes. During response to strong ground-shaking, however, measured responses of structures have shown that the periods lengthen, indicating the loss of the stiffness contributed by those components.

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

5.3.4 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 structure vibrating in each of its important modes. As the structure vibrates back and forth in a particular mode at the associated period, it experiences maximum values of base shear, story 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 W_m for the mode. The coefficient C_{sm} is determined for each mode from Eq. 5.3-3 using the spectral acceleration S_{am} at the associated period of the mode, T_m, in addition to the R, which is discussed elsewhere in the *Commentary*. An exception to this procedure occurs for higher modes of those structures that have periods shorter than 0.3 second and that are founded on soils of Site Class D, E, or F. For such modes, Eq. 5.3-4 is used. Equation 5.3-4 gives values ranging from $0.4S_{DS}/R$ for very short periods to S_{DS}/R for $T_m = 0.3$. Comparing these values to the limiting values of C_s of S_{DS}/R for Site Class D, it is seen that the use of Eq. 5.3-4, when applicable, reduces the modal base shear. This is an approximation introduced in consideration of the conservatism embodied in using the spectral shape specified in Sec. 3.3.4. 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. The design spectrum defined in Sec. 3.3.4 is somewhat more conservative. For Site Classes A, B, and C, the ascending portion of the spectra is completed at or below periods of 0.1 to 0.2 second. On the other hand, for soft soils the ascent may not be completed until a larger period is reached. Equation 5.3-4 is then a replacement for the spectral shape for Site Classes 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. 3.3-5 for modal analysis of structures assigned to Site Classes D, E, and F. The effective modal seismic weight given in Eq. 5.3-2 can be interpreted as specifying the portion of the weight of the structure that participates in the vibration of each mode. It is noted that Eq. 5.3-2 gives values of W_m that are independent of how the modes are normalized.

The final equation of this section, Eq. 5.3-5, is to be used if a modal period exceeds 4 seconds. It can be seen that Eq. 5.3-5 and 5.3-3 coincide at T_m equal to 4 seconds so that the effect of using Eq. 5.3-5 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 $1/T_m$. For very long periods, the average displacement spectrum of strong earthquake motions becomes constant which implies that C_{sm}, a form of acceleration spectrum, should decay as $1/T_m^2$. The period at which the displacement response spectrum becomes constant depends on the size of the earthquake, being larger for great earthquakes, and a representative period of 4 seconds was chosen to make the transition.

5.3.5 Modal forces, deflections, and drifts. This section specifies the forces and displacements associated with each of the important modes of response.

Modal forces at each level are given by Eq. 5.3-6 and 5.3-7 and are expressed in terms of the seismic weight assigned to the floor, the mode shape, and the modal base shear V_m. In applying the forces F_{xm} to the structure, the direction of the forces is controlled by the algebraic sign of f_{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 structure. The form of Eq. 5.3-6 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. 5.2-10 in the ELF procedure.

The modal deflections at each level are specified by Eq. 5.3-8 and 5.3-9. 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. 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 that δ_{xm} is proportional to ϕ_{xm} (this can be shown with algebraic substitution for F_{xm} in Eq. 5.3-9) and will therefore change direction up and down the structure for the higher modes.

5.3.6 Modal story shears and moments. This section merely specifies that the forces of Eq. 5.3-6 should be used to calculate the shears and moments for each mode under consideration. In essence, the forces from Eq. 5.3-6 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 agree with computed using Eq. 5.3-1.

5.3.7 Design values. This section specifies the manner in which the values of story shear, moment, and drift 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 complete quadratic combination (CQC) technique provides somewhat better results than the square-root-of-the-sum-of-the-squares method for the case of closely spaced modes.

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 to be justified because the modal analysis gives a somewhat more accurate representation of the earthquake response. Some limit to the reduction permitted as a result of 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 structural elements not a part of the seismic-force-resisting system and of nonstructural components. The limit is imposed by comparison to 85 percent of the base shear value computed using the ELF procedure. Where modal analysis predicts response quantities corresponding to a total base shear less than 85 percent of that which is computed using the ELF procedure, all response results must be scaled up to that level. Where modal analysis predicts response quantities in excess of those predicted by the ELF procedure, this is likely the result of significant higher mode participation and reduction to the values obtained from the ELF procedure is not permitted.

5.3.8 Horizontal shear distribution. This section requires that the design story shears calculated in Sec. 5.3.6 and the torsional moments prescribed in Sec. 5.2.4 be distributed to the vertical elements of the seismic resisting system as specified in Sec. 5.2.4 and as elaborated on in the corresponding section of this commentary.

5.3.9 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 f_{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. 5.2 (see Sec. 5.2.5 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. 5.2.5 of this commentary.

5.3.10 *P*-delta effects. Sec. 5.2.6 of this commentary applies to this section. In addition, to obtain the story drifts when using the modal analysis procedure of Sec. 5.3, the story drift for each mode should be determined independently for each story. The story drift should not be determined from the differential of combined lateral structural deflections since this latter procedure will tend to mask the higher mode effects in longer period structures.

5.4 LINEAR RESPONSE HISTORY PROCEDURE

Linear response history analysis, also commonly known as time history analysis, is a numerically involved technique in which the response of a structural model to a specific earthquake ground motion accelerogram is determined through a process of numerical integration of the equations of motion. The ground shaking accelerogram, or record, is digitized into a series of small time steps, typically on the order of 1/100th of a second or smaller. Starting at the initial time step, a finite difference solution, or other numerical integration algorithm is followed to allow the calculation of the displacements of each node in the model and the forces in each element of model for each time step of the record. For even small structural models, this requires thousands of calculations and produces tens of thousands of data points. Clearly, such a calculation procedure can be performed only with the aid of high speed computers. However, even with the use of such computers, which are now commonly available, interpretation of the voluminous data that results from such analysis is tedious.

The principal advantages of response history analysis, as opposed to response spectrum analysis, is that response history analysis provides a time dependent history of the response of the structure to a specific ground motion, allowing calculation of path dependent effects such as damping and also providing information on the stress and deformation state of the structure throughout the period of response. A response spectrum analysis, however, indicates only the maximum response quantities and does not indicate when during the period of response these occur, or how response of different portions of the structure is phased relative to that of other portions. Response history analyses are highly dependent on the characteristics of the individual ground shaking records and subtle changes in these records can lead to significant differences with regard to the predicted response of the structure. This is why, when response history analyses are used in the design process, it is necessary to run a suite of ground motion records. The use of multiple records in the analyses allows observation of the difference in response, resulting from differences in record characteristics. As a minimum, the *Provisions* require that suites of ground motions include at least three different records. However, suites containing larger numbers of records are preferable, since when more records are run, it is more likely that the differing response possibilities for different ground motion characteristics are observed. In order to encourage the use of larger suites, the *Provisions* require that when a suite contains fewer than seven records, the maximum values of the predicted response parameters be used as the design values. When seven or more records are used, then mean values of the response parameters may be used. This can lead to a substantial reduction in design forces and displacements and typically will justify the use of larger suites of records.

Where possible, ground motion records should be scaled from actual recorded earthquake ground motions with characteristics (earthquake magnitude, distance from causative fault, and site soil conditions) similar to those which control the design earthquake for the site. Since only a limited number of actual recordings are available for such purposes, the use of synthetic records is permitted and may often be required.

The extra complexity and cost inherent in the use of response history analysis rather than modal response spectrum analysis is seldom justified. As a result this procedure is rarely used in the design process. One exception is for the design of structures with energy dissipation systems comprising linear viscous dampers. Linear response history analysis can be used to predict the response of structures with such systems, while modal response spectrum analysis cannot.

5.5 NONLINEAR RESPONSE HISTORY PROCEDURE

This method of analysis is very similar to linear response history analysis, described in Sec. 5.4, except that the mathematical model is formulated in such a way that the stiffness and even connectivity of the elements can be directly modified based on the deformation state of the structure. This permits the effects of element yielding, buckling, and other nonlinear behavior on structural response to be directly accounted for in the analysis. It also permits the evaluation of such nonlinear behaviors as foundation rocking, opening and closing of gaps, and nonlinear viscous and hysteric damping. Potentially, this ability to directly account for these various nonlinearities can permit nonlinear response history analysis to provide very accurate evaluations of the response of the structure to strong ground motion. However, this accuracy can seldom be achieved in practice. This is partially because currently available nonlinear models for different elements can only approximate the behavior of real structural elements. Another limit on the accuracy of this approach is the fact that minor deviations in ground motion, such as those described in Sec. 5.4, or even in element hysteretic behavior, can result in significant differences in predicted response. For these reasons, when nonlinear response history analysis is used in the design process, suites of ground motion time histories must be considered, as described in Sec. 5.4. It may also be appropriate to perform sensitivity studies, in which the assumed hysteretic properties of elements are allowed to vary, within expected bounds, to allow evaluation of the effects of such uncertainties on predicted response.

Application of nonlinear response history analysis to even the simplest structures requires large, high speed computers and complex computer software that has been specifically developed for this purpose. Several software packages have been in use for this purpose in universities for a number of years. These include the DRAIN family of programs and also the IDARC and IDARST family of programs. However, these programs have largely been viewed as experimental and are not generally accompanied by the same level of documentation and quality assurance typically found with commercially available software packages typically used in design offices. Although commercial software capable of performing nonlinear response history analyses has been available for several years, the use of these packages has generally been limited to complex aerospace, mechanical, and industrial applications.

As a result of this, nonlinear response history analysis has mostly been used as a research (rather than design) tool until very recently. With the increasing adoption of base isolation and energy dissipation technologies in the structural design process, however, the need to apply this analysis technique in the design office has increased, creating a demand for more commercially available software. In response to this demand, several vendors of commercial structural analysis software have modified their analysis programs to include limited nonlinear capability including the ability to model base isolation bearings, viscous dampers, and friction dampers. Some of these programs also have a limited library of other nonlinear elements including beam and truss elements. Such software provides the design office with the ability to begin to practically implement nonlinear response history analysis on design projects. However, such software is still limited, and it is expected that it will be some years before design offices can routinely expect to utilize this technique in the design of complex structures.

5.5.3.1 Member strength. Nonlinear response history analysis is primarily a deformation-based procedure, in which the amount of nonlinear deformation imposed on elements by response to earthquake ground shaking is predicted. As a result, when this analysis method is employed, there is no general need to evaluate the strength demand (forces) imposed on individual elements of the structure. Instead, the adequacy of the individual elements to withstand the imposed deformation demands is directly evaluated, under the requirements of Sec. 5.5.3.2. The exception to this is the requirement to evaluate brittle elements, the failure of which could result in structural collapse, for the forces predicted by the analysis. These elements are identified in the *Provisions* through the requirement that they be evaluated for earthquake forces using the seismic effects defined in Sec. 4.2.2.2. That section requires that forces predicted by elastic analysis be amplified by a factor, Ω_0, to account in an approximate manner for the actual maximum force that can be delivered to the element, considering the inelastic behavior of the structure. Since nonlinear response history analysis does not use a response modification factor, as do elastic analysis approaches, and directly accounts for inelastic structural behavior, there is no need to further increase the forces by this factor. Instead the forces predicted by the analysis are used directly in the evaluation of the elements for adequacy under Sec. 4.2.2.2.

5.5.4 Design review. The provisions for design using linear methods of analysis including the equivalent lateral force technique of Sec. 5.2 and the modal response spectrum analysis technique of Sec. 5.3, are highly prescriptive. They limit the modeling assumptions that can be employed as well as the minimum strength and stiffness the structure must possess. Further, the methods used in linear analysis have become standardized in practice such that it is unlikely that different designers using the same technique to analyze the same structure will produce substantially different results. However, when nonlinear analytical methods are employed to predict the structure's strength and its deformation under load, many of these prescriptive provisions are no longer applicable. Further, as these methods are currently not widely employed by the profession, the standardization that has occurred for linear methods of analysis has not yet been developed for these techniques. As a result analysis has not yet been developed for these techniques, and the designer using such methods must employ a significant amount of independent judgment in developing appropriate analytical models, performing the analysis, and interpreting the results to confirm the adequacy of a design. Since relatively minor changes in the assumptions used in performing a nonlinear structural analysis can significantly affect the results obtained from such an analysis, it is imperative that the assumptions used be appropriate. The *Provisions* require that designs employing nonlinear analysis methods be subjected to independent design review in order to provide a level of assurance that the independent judgment applied by the designer when using these methods is appropriate and compatible with that which would be made by other competent practitioners.

5.6 SOIL-STRUCTURE INTERACTION EFFECTS

5.6.1 General

Statement of the problem. Fundamental to the design requirements presented in Sec. 5.2 and 5.3 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. This assumption implies that the foundation-soil system underlying the structure is rigid and, hence, represents a “fixed-base” condition. Strictly speaking, this assumption never holds in practice. For structures supported on a deformable 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, and soil-structure interaction effects in general, tend to be most significant for laterally stiff structures such as buildings with shear walls, particularly those located on soft soils. For convenience, in what follows the response of a structure supported on a deformable foundation-soil system will be denoted as the “flexible-base” response.

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. 5.6 represent the difference in the flexible-base and fixed-base responses of the structure. This difference depends on the properties of the structure and the supporting medium as well as the characteristics of the free-field ground motion.

The interaction effects accounted for in Sec. 5.6 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. 5.2 and 5.3 and are accounted for only implicitly in Sec. 5.6.

Possible approaches to the problem. Two different approaches may be used to assess the effects of soil-structure interaction. The first involves modifying the stipulated free-field design ground motion, evaluating the response of the given structure to the modified motion of the foundation, and solving simultaneously with additional equations that define the motion of the coupled system, 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 (Jennings and Bielak, 1973; 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 requirements presented in Sec. 5.6.

Characteristics of interaction. The interaction effects in the approach used here are expressed by an increase in the fundamental natural period of the structure and a change (usually an increase) in its effective damping.

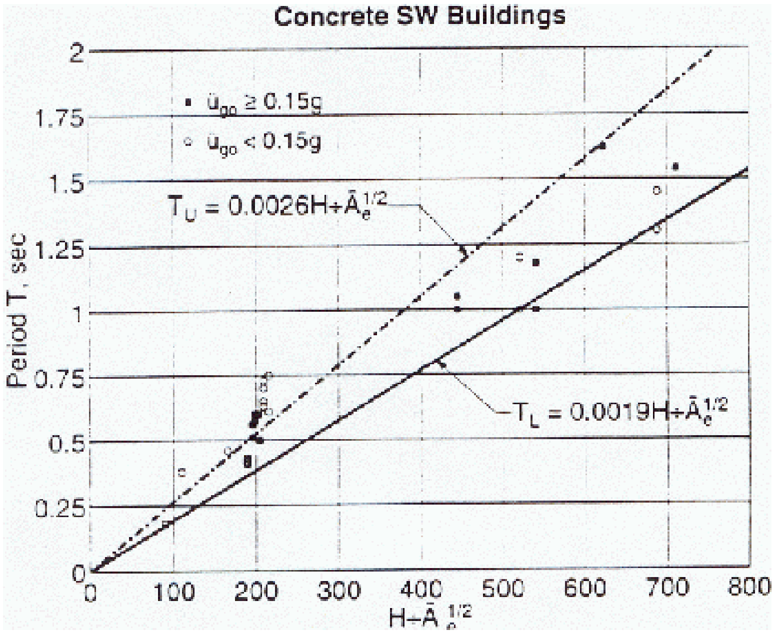

Figure C5.2-3 Measured building period for concrete shear wall structures.

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.

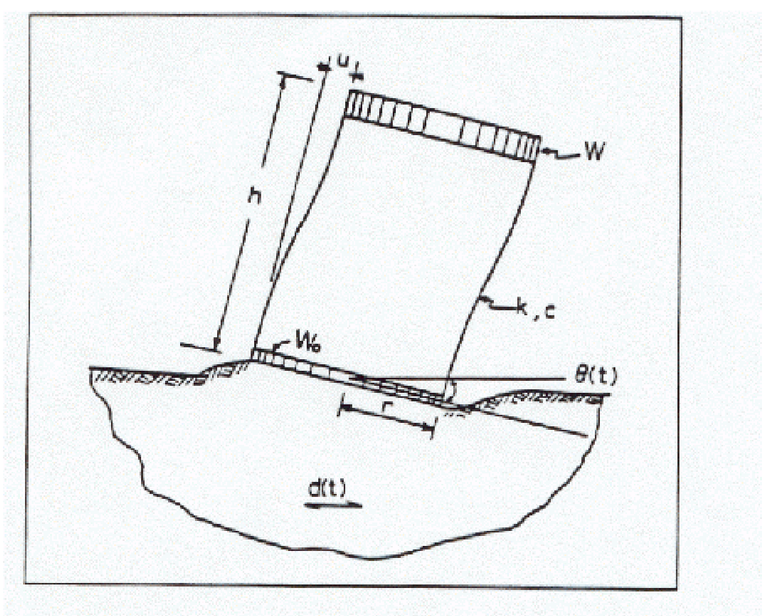

Figure C5.6-1 Simple system investigated.

Consider a linear structure of weight *W*, lateral stiffness *k*, and coefficient of viscous damping *c* (shown in Figure C5.6-1) 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 of the top of the structure, *u*, relative to its base. The system may be viewed either as the direct model of a one-story structural 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 C5.6-2 and C5.6-3 represent response spectra for the steady-state amplitude of the total shear in the columns of the system considered in Figure C5.6-1. 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 $\delta_0 = \dfrac{h}{v_s T}$ 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 C5.6-2 and C5.6-3 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 due to 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 $d_m = \dfrac{a_m T_0^2}{4\pi^2}$.

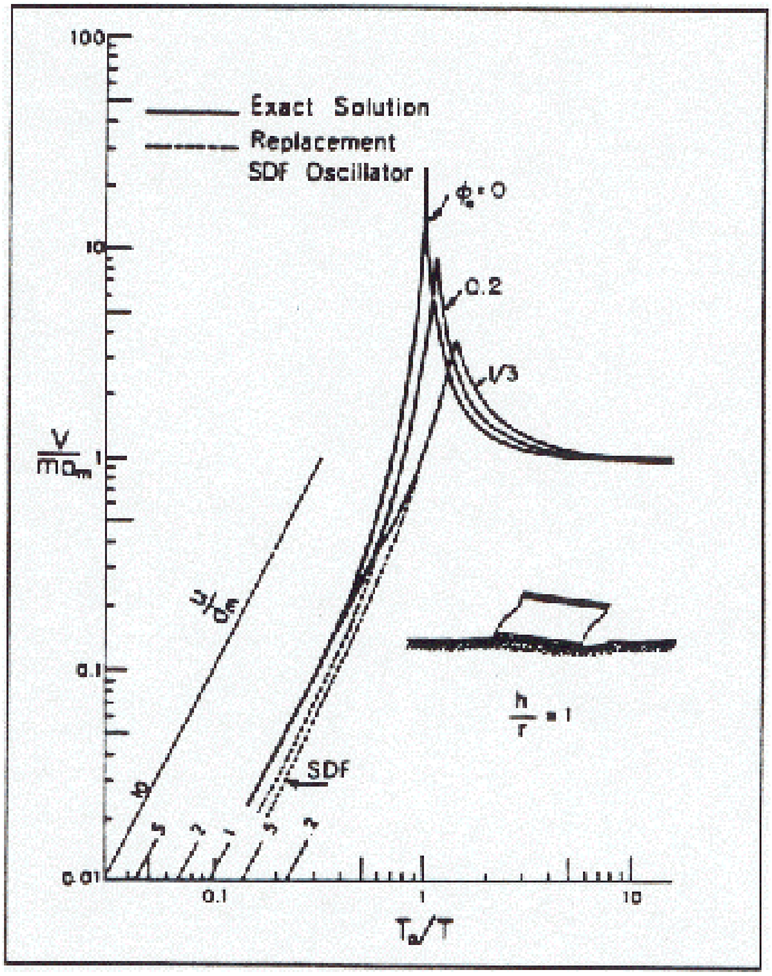

Figure C5.6-2 Response spectra for systems with *h*/*r* = 1 (Veletsos and Meek, 1974).

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

Comparison of the results presented in these figures reveals that the effects of soil-structure interaction are most strikingly reflected in a shift of the peak of the response spectrum to the right and a change in the magnitude of the peak. These changes, which are particularly prominent for taller structures and more flexible soils (increasing values of ϕ_0), 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; Jennings and Bielak, 1973).

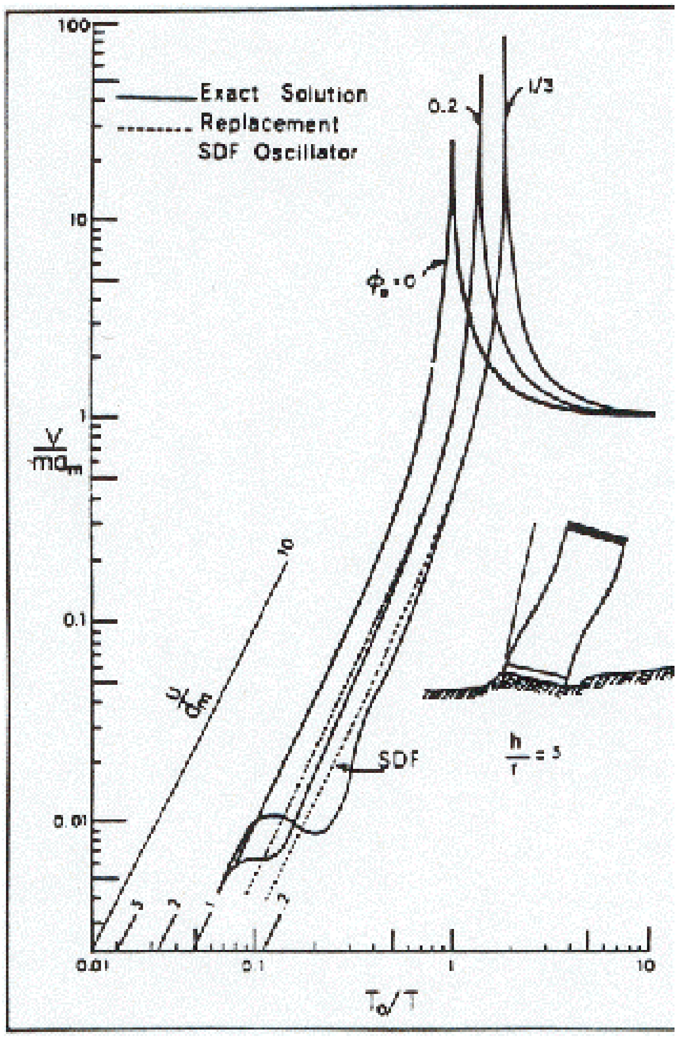

Figure C5.6-3 Response spectra for systems with *h*/*r* = 5 (Veletsos and Meek, 1974).

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. 5.6.2.1.1 and 5.6.2.1.2.

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, the results of such studies may be of limited applicability for foundation systems consisting of individual spread footings or deep foundations (piles or drilled shafts) not interconnected with grade beams or a mat. The requirements presented in Sec. 5.6 for the latter cases represent the best interpretation and judgment of the developers of the requirements regarding the current state of knowledge.

Fundamental to these requirements 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 (for example, to avoid soil instability and settlement associated with the compaction and liquefaction of loose granular soils), is beyond the scope of Sec. 5.6. Finally, no account is taken of the interaction effects among neighboring structures.

Nature of interaction effects. Depending on the characteristics of the structure and the ground motion under consideration, soil-structure interaction may increase, decrease, or have no effect on the magnitudes of the maximum forces induced in the structure itself (Bielak, 1975; Jennings and Bielak, 1973; Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975). However, for the conditions stipulated in the development of the requirements for rigidly supported structures presented in Sec. 5.2 and 5.3, soil-structure interaction will reduce the design values of the base shear and moment from the levels applicable to a fixed-base condition. These forces therefore can be evaluated conservatively without the adjustments recommended in Sec. 5.6.

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

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. 5.2 and involves the use of equivalent lateral static forces. The second is an extension of the simplified modal analysis procedure presented in Sec. 5.3. 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. 5.6. However, it is believed that the more involved procedures are justified only for unusual structures and when the results of the specified simpler approaches have revealed that the interaction effects are indeed of definite consequence in the design.

5.6.2 Equivalent lateral force procedure. This procedure is similar to that used in the older SEAOC recommendations except that it incorporates several improvements (see Sec. 5.2 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 structures, this assumption has been found to be adequate (Bielak, 1976; Jennings and Bielak, 1973; Veletsos, 1977).

5.6.2.1 Base shear. With the effects of soil-structure interaction neglected, the base shear is defined by Eq. 5.2-1, $V = C_s W$, in which W is the total seismic weight (as specified in Sec. 5.2.1) and C_s is the dimensionless seismic response coefficient (as defined in Sec. 5.2.1.1). This term depends on the level of seismic hazard under consideration, the properties of the site, and the characteristics of the structure itself.

The latter characteristics include the rigidly supported 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 Sec. 5.2.1.1 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. 5.2-1 in the form:

$$V = C_s(T,\beta)\bar{W} + C_s(T,\beta)[W - \bar{W}] \tag{C5.6-1}$$

where $\bar{W}$ represents the generalized or effective weight of the structure when vibrating in its fundamental natural mode. The terms in parentheses are used to emphasize the fact that C_s depends upon both T and β. The relationship between $\bar{W}$ and W is given below. The first term on the right side of Eq. C5.6-1 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. C5.6-1) as follows:

$$\tilde{V} = C_s(\tilde{T},\tilde{\beta})\bar{W} + C_s(T,\beta)[W - \bar{W}] \tag{C5.6-2}$$

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. C5.6-2, it is desirable to rewrite this formula in the same form as Eq. 5.6-1. Making use of Eq. 5.2-1 and rearranging terms, the following expression for the reduction in the base shear is obtained:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\tilde{\beta}) \right]\bar{W} \tag{C5.6-3}$$

Within the ranges of natural period and damping that are of interest in studies of structural response, the values of C_s corresponding to two different damping values but the same natural period (T), are related approximately as follows:

$$C_s(\tilde{T},\tilde{\beta}) = C_s(\tilde{T},\beta)\left(\frac{\beta}{\tilde{\beta}}\right)^{0.4} \tag{C5.6-4}$$

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

Substitution of Eq. C5.6-4 in Eq. C5.6-3 leads to:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\beta)\left(\frac{\beta}{\tilde{\beta}}\right)^{0.4} \right]\bar{W} \tag{C5.6-5}$$

where both values of C_s are now for the damping factor of the rigidly supported system and may be evaluated from Eq. 5.2-2 and 5.2-3. 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. C5.6-5 reduces to Eq. 5.6-2.

Note that $\tilde{C}_s$ in Eq. 5.6-2 is smaller than or equal to C_s because Eq. 5.2-3 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. 5.6-10), 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 structure, $\bar{W}$, is defined by Eq. 5.3-2, 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 structures. As an example, it is noted that for a tall structure 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 $\bar{W} = 0.7\ W$. 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.

5.6.2.1.1 Effective building period. Equation 5.6-3 for the effective natural period of the elastically supported structure, $\tilde{T}$, is determined from analyses in which the superstructure is presumed to respond in its fixed-base fundamental mode and the foundation weight is considered to be negligible in comparison to the weight of the superstructure (Jennings and Bielak, 1973; Veletsos and Meek, 1974). The first term under the radical represents the period of the fixed-base structure. The first portion of the second term represents the contribution to $\tilde{T}$ of the translational flexibility of the foundation, and the last portion represents the contribution of the corresponding rocking flexibility. The quantities $\bar{k}$ and $\bar{h}$ represent, respectively, the effective stiffness and effective height of the structure, and K_y and K_θ represent the translational and rocking stiffnesses of the foundation.

Equation 5.6-4 for the structural stiffness, $\bar{k}$, is deduced from the well known expression for the natural period of the fixed-base system:

$$T = 2\pi\sqrt{\left(\frac{1}{g}\right)\left(\frac{\bar{W}}{\bar{k}}\right)} \qquad \text{(C5.6-6)}$$

The effective height, $\bar{h}$, is defined by Eq. 5.6-13, in which ϕ_{i1} has the same meaning as the quantity ϕ_{im} in Eq. 5.3-2 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 structures. As an example, it is noted that for tall structures 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:

$$K_y = \left[\frac{8\alpha_y}{2-v}\right]Gr \qquad \text{(C5.6-7)}$$

and

$$K_\theta = \left[\frac{8\alpha_\theta}{3(1-v)}\right] Gr^3 \qquad \text{(C5.6-8)}$$

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

$$G = \frac{\gamma v_s^2}{g} \qquad \text{(C5.6-9)}$$

in which γ is the unit weight of the material. The values of G, v_s, and v should be interpreted as average values for the region of the soil that is affected by the forces acting on the foundation and should correspond to the conditions developed during the design earthquake. The evaluation of these quantities is considered further in subsequent sections. For statically loaded foundations, the stiffness coefficients α_y and α_θ are unity, and Eq. C5.6-7 and C5.6-8 reduce to:

$$K_y = \frac{8Gr}{2-v} \qquad \text{(C5.6-10)}$$

and

$$K_\theta = \frac{8Gr^3}{3(1-v)} \qquad \text{(C5.6-11)}$$

Studies of the interaction effects in structure-soil systems have shown that, within the ranges of parameters of interest for structures subjected to earthquakes, the results are insensitive to the period-dependency of α_y and that it is sufficiently accurate for practical purposes to use the static stiffness K_y, defined by Eq. C5.6-10. However, the dynamic modifier for rocking α_θ can significantly affect the response of building structures. In the absence of more detailed analyses, for ordinary building structures with an embedment ratio $d/r < 0.5$, the factor α_θ can be estimated as follows:

R/v_sT	α_θ
<0.05	1.0
0.15	0.85
0.35	0.7
0.5	0.6

where d equals depth of embedment and r can be taken as r_m defined in Eq. 5.6-8.

The above values were derived from the solution for α_θ by Veletsos and Verbic (1973). In this solution α_θ is a function of $\tilde{T}$. To relate α_θ to T, a correction for period lengthening $(\tilde{T}/T)$ was made assuming $\bar{h}/r \sim 0.5 \text{ to } 1.0$ and Poisson's ratio $v = 0.4$.

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:

$$K_y = \left[\frac{8Gr}{2-v}\right]\left[1+\left(\frac{2}{3}\right)\left(\frac{d}{r}\right)\right] \qquad \text{(C5.6-12)}$$

and

$$K_\theta = \left[\frac{8Gr^3\alpha_\theta}{3(1-v)}\right]\left[1+2\left(\frac{d}{r}\right)\right] \tag{C5.6-13}$$

in which d is the depth of embedment. These formulas are based on finite element solutions (Kausel, 1974).

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. C5.6-12 and C5.6-13. 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. C5.6-12 and C5.6-13 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 surface stratum of soil underlain by a stiffer deposit with a shear wave velocity (v_s) more than twice that of the surface layer (Wallace et al., 1999), K_y and K_θ may be determined from the following 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. C5.6-12:

$$K_y = \left[\frac{8Gr}{2-v}\right]\left[1+\left(\frac{2}{3}\right)\left(\frac{d}{r}\right)\right]\left[1+\left(\frac{1}{2}\right)\left(\frac{r}{D_s}\right)\right]\left[1+\left(\frac{5}{4}\right)\left(\frac{d}{D_s}\right)\right] \tag{C5.6-14}$$

Second, using Eq. C5.6-13:

$$K_\theta = \left[\frac{8Gr^3\alpha_\theta}{3(1-v)}\right]\left[1+2\left(\frac{d}{r}\right)\right]\left[1+\left(\frac{1}{6}\right)\left(\frac{r}{D_s}\right)\right]\left[1+0.7\left(\frac{d}{D_s}\right)\right] \tag{C5.6-15}$$

These formulas are based on analyses of a stratum supported on a rigid base (Elsabee et al., 1977; Kausel and Roesset, 1975) and apply for $r/D_s < 0.5$ and $d/r < 1$.

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 is replaced by r_a (Eq. 5.6-7), 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 is replaced by r_m (Eq. 5.6-8), 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:

$$K_y = \Sigma k_{yi} \tag{C5.6-16}$$

and

$$K_\theta = \Sigma k_{xi} y_i^2 + \Sigma k_{\theta i} \tag{C5.6-17}$$

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:

$$k_{yi} = \left(\frac{8G_i r_{ai}}{2-v}\right)\left(1+\frac{2}{3}\frac{d}{r}\right) \tag{C5.6-18}$$

$$k_{yi} = \left(\frac{4G_i r_{ai}}{1-v}\right)\left(1+0.4\frac{d}{r}\right) \tag{C5.6-19}$$

and

$$k_{\theta i} = \left(\frac{8G_i r^3{}_{mi}}{3(1-v)}\right)\left(1+2\frac{d}{r}\right) \tag{C5.6-20}$$

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

For surface-supported footings and for embedded footings for which the side wall contact with the soil cannot be considered to be effective during the stipulated design ground motion, d_i in these formulas should be taken as zero. Furthermore, the values of G_i should be consistent with the stress levels expected under the footings and should be evaluated with due regard for the effects of the dead loads involved. This matter is considered further in subsequent sections. For closely spaced footings, consideration of the coupling effects among footings will reduce the computed value of the overall foundation stiffness. This reduction, in turn, will increase the fundamental natural period of the system, $\tilde{T}$, and increase 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. C5.6-16 and 5.6-17 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. C5.6-16 and C5.6-17.

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 (Tomlinson, 1994) that express these stiffnesses in terms of the modulus of the subgrade reaction and the properties of the pile itself. These stiffnesses sometimes 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.

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, v. 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 $0.75r_a$ below the foundation base for horizontal motions, $2r_a$ for vertical motions, and to about $0.75r_m$ for rocking motion. For mat foundations, the effective depth is related to the total plan dimensions of the mat whereas for structures 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 5.6-1. 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. C5.6-9. 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, 1978), 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 foundation may be expressed as the sum of two terms as follows:

$$\bar{\sigma}_o = \bar{\sigma}_{os} + \bar{\sigma}_{ob} \qquad \text{(C5.6-21)}$$

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 structure and foundation. The first term is defined by the formula:

$$\bar{\sigma}_{os} = \left(\frac{1+2K_o}{3}\right)\gamma' x \qquad \text{(C5.6-22)}$$

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, $K_o \simeq 1.0$. The pressures $\bar{\sigma}_{ob}$ developed by the weight of the structure 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:

$$v_{so} = c_1(2.17 - e)(\bar{\sigma})^{0.25} \qquad \text{(C5.6-23)}$$

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:

$$v_{so} = c_2(2.97 - e)(\bar{\sigma})^{0.25} \qquad \text{(C5.6-24)}$$

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 C5.6-24 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:

$$G_o = 1{,}000 s_u \qquad \text{(C5.6-25)}$$

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, 1975).

These empirical relations may be used to obtain preliminary, order-of-magnitude estimates. For more accurate evaluations, field measurements of v_{so} should be made. Field evaluations of the variations of v_{so}

throughout the construction site can be carried out by standard seismic refraction methods, the downhole or cross-hole methods, suspension logging, or spectral analysis with surface waves. Kramer (1996) provides an overview of these testing procedures. The disadvantage of these methods 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. C5.6-22, C5.6-23, and C5.6-24 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 5.6-1. This matter is considered further in the next two sections.

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 (for example, Vucetic and Dobry, 1991; Seed et al., 1984) 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 5.6-1. For each severity of anticipated ground-shaking, represented by the effective peak acceleration coefficients (taken as $0.4S_{DS}$) 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 5.6-1 are first order approximations. More precise evaluations would require the use of material-specific shear modulus reduction curves 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: v = 0.33 for clean sands and gravels, v = 0.40 for stiff clays and cohesive soils, and v = 0.45 for soft clays. The use of an average value of v = 0.4 also will be adequate for practical purposes.

Regarding an alternative approach, note that Eq. 5.6-5 for the period $\tilde{T}$ of structures supported on mat foundations was deduced from Eq. 5.6-3 by making use of Eq. C5.6-10 and C5.6-11, with Poisson's ratio taken as v = 0.4 and with the radius r interpreted as r_a in Eq. C5.6-10 and as r_m in Eq. C5.6-11. For a nearly square foundation, for which $r_a \simeq r_m \simeq r$, Eq. 5.6-5 reduces to:

$$\tilde{T} = T\sqrt{1 + 25\alpha\left(\frac{r\bar{h}}{v_s^2 T^2}\right)\left[1 + \left(\frac{1.12\bar{h}^2}{\alpha_\theta r^2}\right)\right]} \tag{C5.6-26}$$

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

5.6.2.1.2 Effective damping. Equation 5.6-9 for the overall damping factor of the elastically supported structure, $\tilde{\beta}$, was determined from analyses of the harmonic response at resonance of simple systems of the type considered in Figures C5.6-2 and 5.6-3. The result is an expression of the form (Bielak, 1975; Veletsos and Nair, 1975) of:

$$\tilde{\beta} = \beta_0 + \frac{0.05}{\left(\frac{\tilde{T}}{T}\right)^3} \tag{C5.6-27}$$

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 C5.6-27 corresponds to the value of β = 0.05 used in the development of the response spectra for rigidly supported systems employed in Sec. 5.2.

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 (that is, 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 5.6-1 for two levels of excitation. The dashed lines, which are recommended for values of the effective peak ground acceleration (taken as $0.4S_{DS}$) equal to or less than 0.10, correspond to a value of $\Delta W_s/W_s \simeq 0.3$, whereas the solid lines, which are recommended for values of effective peak ground acceleration equal to or greater than 0.20, correspond to a value of $\Delta W_s/W_s \simeq 1$. 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 5.6-1 should be interpreted as a characteristic length that is related to the length of the foundation, L_o, in the direction in which the structure is being analyzed. For short, squatty structures for which $\bar{h}/L_o \leq 0.5,$ the overall damping of the structure-foundation system is dominated by the translational action of the foundation, and it is reasonable to interpret r as r_a, the radius of a disk that has the same area as that of the actual foundation (see Eq. 5.6-7). On the other hand, for structures with $\bar{h}/L_o \geq 0.1,$ 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. 5.6-8).

Subject to the qualifications noted in the following section, the curves in Figure 5.6-1 also may be used for embedded mat foundations and for foundations involving spread footings or piles. In the latter cases, the quantities A_o and I_o in the expressions for the characteristic foundation length, r, should be interpreted as the area and the moment of inertia of the load-carrying foundation.

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

There is one additional source of conservatism in the application of the recommended requirements to structures 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. 5.6 and in the requirements for rigidly supported structures presented in Sec. 5.2 and 5.3.

Equations 5.6-9 and C5.6-28, in combination with the information presented in Figure 5.6-1, may lead to damping factors for the structure-soil system, $\tilde{\beta},$ that are smaller than the structural damping factor, β. However, since the representative value of $\beta = 0.05$ used in the development of the design requirements for rigidly supported structures is based on the results of tests on actual structures, 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. 5.6-9 should never be taken less than β, and a minimum value 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 5.6-1.

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

$$T_s = \frac{4D_s}{v_s} \tag{C5.6-28}$$

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 5.6-1 are applicable only when:

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} \geq 1 \tag{C5.6-29}$$

For

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} < 1 \tag{C5.6-30}$$

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. 5.6-10.

For $T_s / \tilde{T} = 1$, Eq. 5.6-10 leads to $\beta_o' = \beta_o$ whereas for $T_s / \tilde{T} = 0$, it leads to $\beta_0' = 0$, 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_0' 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.

5.6.2.2 and 5.6.2.3 Vertical distribution of seismic forces and other effects. The vertical distributions of the equivalent lateral forces for flexibly and rigidly supported structures are generally different. However, the differences are inconsequential for practical purposes, and it is recommended that the same distribution be used in both cases, changing only the magnitude of the forces to correspond to the appropriate base shear. A greater degree of refinement in this step would be inconsistent with the approximations embodied in the requirements 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.

5.6.3 Response spectrum 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 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 equivalent lateral force procedure, 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:

$$\bar{W} = \bar{W}_1 = \frac{\left(\Sigma w_i \phi_{i1}\right)^2}{\Sigma w_i \phi_{i1}^2} \tag{C5.6-31}$$

which is the same as Eq. 5.3-2, and $\bar{h}$ is computed from Eq. 5.6-13. The quantity ϕ_{i1} in these formulas represents the displacement amplitude of the i^{th} floor level when the structure is vibrating in its fixed-base, fundamental natural mode. The structural stiffness, $\bar{k}$, is obtained from Eq. 5.6-4 by taking $\bar{W} = \bar{W}_1$ and using for T the fundamental natural period of the fixed-base structure, T_1. The fundamental natural period of the interacting system, $\tilde{T}_1$, is then computed from Eq. 5.6-3 (or Eq. 5.6-5 when applicable) by taking $T = T_1$. The effective damping in the first mode, β, is determined from Eq. 5.6-9 (and Eq. 5.6-10 when applicable) in combination with the information given in Figure 5.6-1. The quantity $\bar{h}$ in the latter figure is computed from Eq. 5.6-13.

With the values of $\tilde{T}_1$ and $\tilde{\beta}_1$ established, the reduction in the base shear for the first mode, ΔV_1, is computed from Eq. 5.6-2. The quantities C_s and $\tilde{C}_s$ in this formula should be interpreted as the seismic coefficients corresponding to the periods T_1 and $\tilde{T}_1$, respectively; $\tilde{\beta}$ should be taken equal to $\tilde{\beta}_1$; and $\bar{W}$ should be determined from Eq. C5.6-31.

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. 5.6-14 represents the contribution of the foundation rotation.

5.6.3.3 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 requirements of Sec. 5.2.4 and the P-delta effects should be evaluated in accordance with the requirements of Sec. 5.2.6.2, using the story shears and drifts determined in Sec. 5.6.3.2.

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 and only when the requirements indicate that the interaction effects are of definite consequence in design, would the use of more elaborate procedures be justified. Some of the possible refinements, listed in order of more or less increasing complexity, are:

1. Improve the estimates of the static stiffnesses of the foundation, K_y and K_θ, and of the foundation damping factor, β_0, by considering in a more precise manner the foundation type involved, the effects of foundation embedment, variations of soil properties with depth, and hysteretic action in the soil. Solutions may be obtained in some cases with analytical or semi-analytical formulations and in others by application of finite difference or finite element techniques. A concise review of available analytical formulations is provided in Gazetas (1991). 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 or 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 (for example, Lysmer et al., 1981; Borja et al., 1992), taking due account of the nonlinear effects in both the structure and the supporting medium.

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

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Appendix to Chapter 5

NONLINEAR STATIC PROCEDURE

A5.2 NONLINEAR STATIC PROCEDURE

The nonlinear static procedure is intended to provide a simplified approach for directly determining the nonlinear response behavior of a structure at different levels of lateral displacements, ranging from initial elastic response through development of a failure mechanism and initiation of collapse. Response behavior is gauged by measurement of the strength of the structure, at various increments of lateral displacement.

Usually the shear resisted by the system at yield of the first element of the structure is defined as the "elastic strength," although this may not correspond to yield of the entire structure. When traditional linear methods of design (using *R* factors) are employed, this elastic strength will not be less than the design base shear.

If a structure is subjected to larger lateral loads than that represented by the elastic strength, a number of elements will yield—eventually forming a mechanism. For most structures, multiple configurations of mechanisms are possible. The mechanism caused by the smallest set of forces is likely to appear before others do. That mechanism is considered to be the dominant mechanism. Standard methods of plastic or "limit" analysis can be used to determine the strength corresponding to such mechanisms. However, such "limit analysis" cannot determine the deformation at the onset of such a mechanism. If the yielding elements are able to strain harden, the mechanism will not allow an increase of deformations without some increase of lateral forces and the mechanism is stable. Moreover, it can be considered as a flexible version of the original frame structure. Figure CA5.2-1, which shows a plot of the lateral structural strength vs. deformation (or pushover curve) for a hypothetical structure, illustrates these concepts.

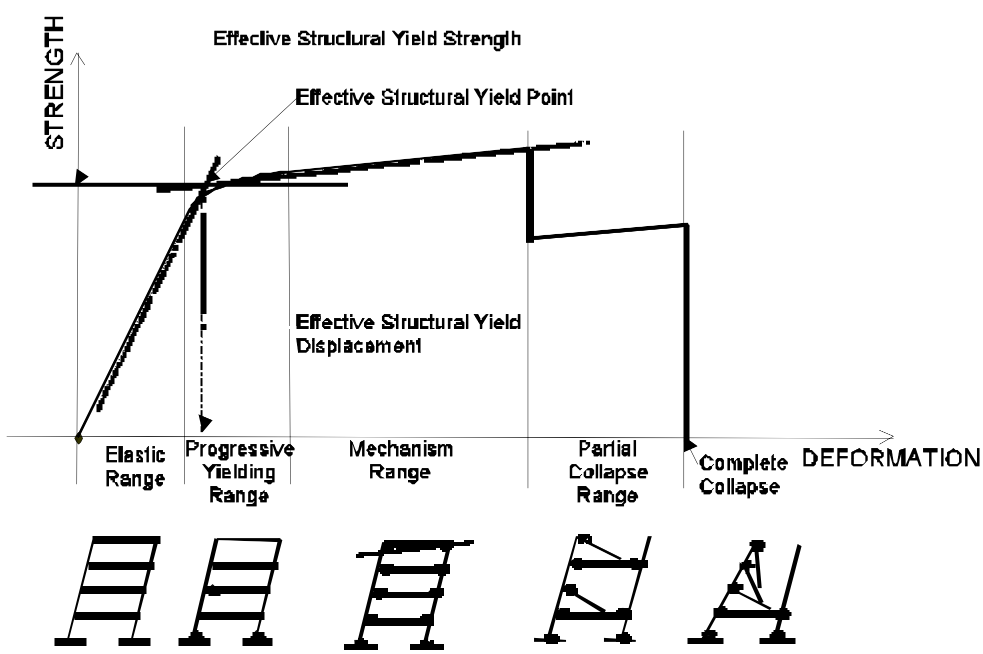

Figure CA5.2-1 Strength-deformation relation for a frame structure

If, after the structure develops a mechanism, it deforms an additional substantial amount, elements within the structure may fail (fracture, buckle, etc.) and thus cease to contribute strength to the structural system. In such cases, the strength of the structure will diminish with increasing deformation. In the event of failure of an essential element, or group of elements, the entire structure may lose capacity to carry the gravity or lateral loads. Such loss of load-carrying capacity can also occur if the lateral deformation becomes so great that the *P*-delta effects exceed the residual lateral stiffness of the structure. Such conditions are defined as collapse and the deformation associated with collapse is defined as the "ultimate deformation." This deformation can be determined by the nonlinear static procedure and also by plastic or limit analysis.

As shown in Figure CA5.2-1, many structures exhibit a range of behavior between the development of first yielding and development of a mechanism. When the structure deforms while elements are yielding sequentially (shown as progressive yielding), the relation between external forces and deformations cannot be determined by simple limit analysis. For such a case, other methods of analysis are required. The purpose of nonlinear static procedure is to provide a simplified method of determining structural response behavior at deformation levels between those that can be conveniently analyzed using limit state methods.

A5.2.1 Modeling. In this procedure, the structure is modeled using elements having stiffness properties that are dependent on the amount of deformation imposed on the element. All elements than can be subjected to deformations or forces larger than those corresponding to yield should be modeled with nonlinear properties. At a minimum, nonlinear stiffness properties (using a bilinear model) should include initial elastic stiffness, yield strength (and yield deformation), and post-yield characteristics including the point of loss of strength (and associated deformation) or point of complete fracture or loss of stability.

A5.2.2 Lateral loads. The analysis is performed by applying a incrementally increasing pattern of lateral loads distributed throughout the structure. The analysis traces the internal distribution of loads and deformations as the load amplitude is progressively increased. Moreover it records the strength-deformation relation and the characteristic events occurring as the analysis progresses. The strength-deformation relation typically takes a shape similar to that shown in Figure CA5.2-1.

It should be noted that nonlinear static analysis can be used to determine the order of yielding of elements in the "progressive yielding range" (see Figure CA5.2-1) and the associated strengths and deformations. The analysis can also identify the deformations associated with fractures or failure of components and the entire structure. However, it is accurate only if the applied pattern of loads induces a pattern of deformation in the structure that is similar to that which will be induced by the earthquake ground motion. This can be controlled, to some extent, through application of an appropriate pattern of loads. However, this method is generally limited in applicability to structures that have limited higher-mode participation.

The force-deformation sequence predicted by the analysis is a function of the configuration of the set of monotonically increasing loads. In order to capture the dynamic behavior of the structure, the force-deformation relation should be properly defined as the instantaneous distribution of inertial forces when the maximum response of structure occurs. Therefore, the load configuration should be redefined at each point on the pushover curve, proportional to the instantaneous configuration of inertial forces. Such a configuration is dependent on the instantaneous modal characteristics of the structure and their combination. Since the structure is nonlinear, the instantaneous modal characteristics depend on the modified properties due to inelastic deformations, affecting the load distribution at each step, accordingly.

Such use of a varying, deformation-dependent load configuration would require almost as much labor and uncertainty as application of a full nonlinear response history procedure. Such effort would be inappropriate for the simplified approach that the nonlinear static procedure is intended to provide. Therefore, the load configuration and intensity are approximated in the nonlinear static procedures. Several approximations are available, including the following:

1. An approximate distribution proportional to the idealized elastic response model as used in the equivalent lateral force procedure:

$$F_i = \frac{w_i h_i^k}{\sum_j w_j h_i^k} V \tag{CA5.2-1}$$

where, F, w, h and V are the story inertia force, story weight, story height, and base shear, respectively; k is a coefficient ranging between 1 and 2, as defined in *Provisions* Sec. 5.2.3.

2. A better approximation, using the dominant mode of vibration (such as the first mode in moderate height building structures):

$$F_i = \frac{w_i \phi_i}{\sum_i w_i \phi_i} V \tag{CA5.2-2}$$

where, ϕ_i is the dominant mode shape. This approximation allows the three-dimensional distribution of inertia forces to be obtained when such considerations are important.

3. A still more complete approximation using several significant modes of vibration. In such cases the modes for which the total equivalent modal mass exceed 90 percent should be included. The load configuration is given by:

$$F_i = \frac{w_i \phi_{id}}{\Sigma w_i \phi_{id}} \frac{\left[\Sigma\left[\left(\Gamma_i/\Gamma_d\right)\left(S_{ai}/S_{ad}\right)\right]^2\right]^{1/2}}{\left[\Sigma\left[\left(\Gamma_i/\Gamma_d\right)^2\left(S_{ai}/S_{ad}\right)\right]^2\right]^{1/2}} V \tag{CA5.2-3}$$

where, Γ_i and S_{ai} are the modal participation factor and the spectral acceleration, respectively, and subscript d indicates the dominant mode. ($\Gamma_i = \Sigma w_i \phi_i$; where the mode shapes, ϕ, are mass normalized—that is $\sum w_i \phi_i^2 / g = 1$.)

4. An approximation that takes into account both higher mode contributions and changes in the loading due to yielding of the structure. In this case the load configuration described by Eq. CA5.2-3) is calculated and reevaluated when the modal characteristics of the structure change as it yields. Such procedure has also termed an "adaptive push-over analysis."

The *Provisions* adopt the simplest of these approaches, indicated as item 1 above, though use of the more complex approaches is not precluded. Nonlinear static analysis options exist in several commercially available and public-domain analysis platforms.

A5.2.3 Target displacement. The nonlinear analysis should be continued by increasing the amplitude of the pattern of lateral loads until the deflections at the control point exceeds 150 percent of the target displacement. The expected inelastic deflection at each level shall be determined by combining the elastic modal values as obtained from Sec. 5.3.5 and 5.3.6 multiplied by the factor

$$C = \frac{\left(1 - T_s/T_1\right)}{R_d} + \left(T_s/T_1\right) \tag{CA5.2-4}$$

where T_s is the characteristic period of the response spectrum, defined as the period associated with the transition from the constant-acceleration segment of the spectrum to the constant-velocity segment of the spectrum and R_d is the ratio of the total design base shear to the fully yielded strength of the major mechanism, which can be obtained according to $R_d = R/\Omega_o$, with R and Ω_o given in Table 4.3-1. The combination shall be carried out by taking the square root of the sum of the squares of each of the modal values or by the complete quadratic combination technique.

The recommendation linking the expected inelastic deformation to the elastic is based on an approach originally suggested by Newmark and on later studies by several other researchers. These are described below.

In a 1991 study, Nassar and Krawinkler published simplified expressions that were derived from a study of mean strength reduction factors computed from fifteen ground motions recorded in the Western United States. The records used were obtained at alluvium and rock sites. The influence of the site conditions was not explicitly considered. The sensitivity of mean strength reduction factors to epicentral distance, yield level, strain-hardening ratio, and stiffness degradation was examined. The study concluded that epicentral distance and stiffness degradation have negligible influence on strength reduction factors and proposed the following relationship for the ratio of inelastic displacements to displacements predicted by elastic analysis:

$$R_d = \left[1 + \frac{1}{c}\left(r^c - 1\right)\right] / r \geq 1 \qquad \text{(CA5.2-5)}$$

where,

$$c = \frac{T^a}{1+T^a} + \frac{b}{T} \qquad \text{(CA5.2-6)}$$

In the above, T, is the period of vibration of the structure and r is the strength ratio. R_d is defined above.

In 1994, Chang and Mander performed analytical studies based on an envelope of five recorded ground motions. The following inelastic dynamic magnification factor that relates the maximum inelastic displacement to the elastic spectral displacement was obtained.

$$R_D = \left(1 - \frac{1}{r}\right)\left(\frac{T_{PV}}{T}\right)^n + \frac{1}{r} \cdot \geq 1 \qquad \text{(CA5.2-7)}$$

where T_{PV} is the period at which the maximum spectral velocity response occurs, and

$$n = 1.2 + 0.025r \text{ for } T_{PV} \leq 1.2 \text{ sec.} \qquad \text{(CA5.2-8)}$$

$$n = 1.2 \text{ for } T_{PV} > 1.2 \text{ sec.} \qquad \text{(CA5.2-9)}$$

In 1992, Vidic, Fajfar, and Fischinger recommended simplified expressions derived from the study of the mean strength reduction factors computed from twenty ground motions recorded in the Western United States as well as in the 1979 Montenegro, Yugoslavia, earthquake. Systems with bilinear and stiffness degrading (Q-model) hysteric behavior and viscous damping proportional to the mass and the instantaneous stiffness were considered, resulting in the following expression:

$$R_D = \left(1 - \frac{1}{r}\right)\frac{T_0}{T} + \frac{1}{r} \geq 1 \qquad \text{(CA5.2-10)}$$

where T is the dominant period of structure, $T_0 = 0.65\mu^{0.3}T_1$, and

$$T_1 = 2\pi \frac{\phi_{ev}}{\phi_{ea}} \frac{V}{A} \qquad \text{(CA5.2-11)}$$

where V and A are the peak ground velocity and peak ground acceleration, respectively. For the 20 ground motions considered in the study, the mean amplification factors ϕ_{ea} and ϕ_{ev} are 2.5 and 2.0, respectively.

Miranda and Bertero (1994) suggested simplified expressions derived from the study of the mean strength reduction factors computed from 124 ground motions recorded on a wide range of soil conditions. The study considered 5-percent-damped bilinear systems undergoing displacement ductility ratios between 2 and 6. Based on the local site conditions at the recording station, ground motions were classified into three groups: rock sites, and soft soil sites. In addition to the influence of soil conditions, the study considered the influence of magnitude and epicentral distance on strength reduction factors. The study concluded that soil conditions influence the reduction factors significantly (particularly for soft soil sites) and that magnitude and epicentral distance have a negligible effect on mean strength reduction factors. The study produced the following expression for the mean strength reduction factor:

$$R_D = \left(1 - \frac{1}{r}\right)\Phi + \frac{1}{r} \qquad \text{(CA5.2-12)}$$

with,

$$\Phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T}\exp\left[-\frac{3}{2}\left(\ln T - \frac{3}{5}\right)^2\right] \qquad \text{(CA5.2-13)}$$

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T}\exp\left[-2\left(\ln T - \frac{1}{5}\right)^2\right] \qquad \text{(CA5.2-14)}$$

(CA5.2-15)

where T is the period of vibration of the structure and T_g is the characteristic ground motion period.

The recommended formulation contained in the *Provisions* is a combination of the recommendations of Krawinkler et al and of Vidic et al with some simplification. The *Provisions* require that the analysis be continued until the deflection at the control point exceeds 150 percent of the target displacement in order to account for inaccuracy due to this simplification and because small variations in strength (due to modeling or due to imprecise construction) can lead to large displacement variations in the inelastic range.

A5.2.5 Design review. See *Commentary* Sec. 5.5.4.

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Chapter 6 Commentary

ARCHITECTURAL, MECHANICAL, AND ELECTRICAL COMPONENT DESIGN REQUIREMENTS

6.1 GENERAL

6.1.1 Scope. The general requirements establish minimum design levels for architectural, mechanical, electrical, and other nonstructural systems and components recognizing occupancy use, occupant load, need for operational continuity, and the interrelation of structural and architectural, mechanical, electrical, and other nonstructural components. Several classes of components are not subject to the *Provisions*:

1. All components in Seismic Design Category A are exempted because of the lower seismic input for these items.
2. All mechanical and electrical components in Seismic Design 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 Design Categories, weighing less than 400 pounds (1780 N), and 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 fall.

Storage racks are considered nonbuilding structures and are covered in *Provisions* Chapter 14. See *Commentary* Sec. 14.3.5.

Storage tanks are considered nonbuilding structures and are covered in *Provisions* Chapter 14. See *Commentary* Sec. 14.4.7.

When performing seismic design of nonstructural components, be aware that there may be important non-seismic requirements outside the scope of the building code, that may be affected by seismic bracing. For example, thermal expansion is often a critical design consideration in pressure piping systems, and bracing must be arranged in a manner that accommodates thermal movements. The design for seismic loads should not compromise the functionality, durability, or safety of the overall system, and this may require substantial collaboration and cooperation between the various disciplines in the design team. In some cases, such as essential facilities or hazardous environments, it may be appropriate to consider performance levels higher than what is required by the building code (for example, operability of a piping system, rather than leak tightness).

For some components, such as exterior walls, the wind design forces may be higher than the seismic design forces. Even when this occurs, the seismic detailing requirements may still govern the overall structural design. Whenever this is a possibility, it should be investigated early in the structural design process.

6.2 GENERAL DESIGN REQUIREMENTS

6.2.2 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. Use of higher I_p requirements together with application of the requirements in Sec. 6.4.2 and 6.4.3 should provide better, more functional component. While this approach will provide a higher degree of confidence in the probable seismic performance of a component, it may not be sufficient for all components. For example, individual ceiling tiles may still fall from the ceiling grid. Seismic qualification approaches presently in use by the Department of Energy (DOE) and the Nuclear Regulatory Commission (NRC) should be considered by the registered design professional and/or the owner when the consequences of failure would be unacceptable.

Components that could fall from the structure are among the most hazardous building components in an earthquake. These components may not be integral with the structural system and may cantilever horizontally or vertically from their supports. Critical issues affecting these components include their weight, their attachment to the structure, their breakage characteristics (glass) and their location (over an entry or exit, public walkway, atrium, or lower adjacent structure). Examples of items that may pose a falling hazard include parapets, cornices, canopies, marquees, glass, and precast concrete cladding panels. In addition, mechanical and electrical components may pose a falling hazard (for example, a rooftop tank or cooling tower, which if separated from the structure would fall to the ground).

Special consideration should be given to components that could block means of egress or exitways if they were to fall during an earthquake. The term “means of egress” has been defined in the same way throughout the country, since egress requirements have been included in building codes because of fire hazard. The requirements for exitways include intervening aisles, doors, doorways, gates, corridors, exterior exit balconies, ramps, stairways, pressurized enclosures, horizontal exits, exit passageways, exit courts, and yards. Example items that should be included when considering egress include walls around stairs and corridors, and veneers, cornices, canopies, and other ornaments above building exits. In addition, heavy partition systems vulnerable to failure by collapse, ceilings, soffits, light fixtures, or other objects that could fall or obstruct a required exit door or component (rescue window or fire escape) could be considered major obstructions. Examples of components that do not pose a significant falling hazard include fabric awnings and canopies and architectural, mechanical, and electrical components which, if separated from the structure, will fall in areas that are not accessible (in an atrium or light well not accessible to the public, for instance).

In Sec. 1.2.1 the intent is that Group III structures shall, in so far as practical, be provided with the capacity to function after an earthquake. To facilitate this, all nonstructural components and equipment in structures in Seismic Use Group III, and in Seismic Design Category C or higher, should be designed with an I_p equal to 1.5. All components and equipment are included because damage to vulnerable unbraced systems or equipment may disrupt operations following an earthquake, even if they are not “life-safety” items. Nonessential items can be considered “black boxes.” There is no need for component analysis as discussed in Sec. 6.4.2 and 6.4.3, since operation of these secondary items is not critical to the post-earthquake operability of the structure. Instead, the design may focus on their supports and attachments.

6.2.3 Consequential damage. Although the components included in Tables 6.3-1 and 6.4-1 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 6.3-1 or 6.4-1 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 non-load-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.

6.2.4 Flexibility. In the design and evaluation of support structures and the attachment of architectural components, flexibility should be considered. Components that are subjected to seismic relative displacements (that is, components that are connected to both the floor and ceiling level above) should be designed with adequate flexibility to accommodate imposed displacements. This is covered in Sec. 6.2.7. 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 effects of flexibility in the load path of the components should be considered especially in the region near the anchor points.

6.2.5 Component force transfer. It is required that components be attached to the structure and that all the required attachments be fully detailed in the design documents, or be specified in accordance with approved standards. These details should take into account the force levels and anticipated deformations expected or designed into the structure. For the purposes of the load path check, it is essential that detailed information concerning the components, including size, weight, and location of component anchors, be communicated to the registered design professional responsible for the structure during the design process.

The calculation of forces as prescribed in Sec. 6.2.6 recognizes the unique dynamic and structural characteristics of the components as compared to structures. Components typically lack the desirable attributes of structures (such as ductility, toughness, and redundancy) that permit the use of greatly reduced lateral design forces. This is reflected in the lower values for R_p given in Tables 6.3-1 and 6.4-1, as compared to R values for structures. In addition, components may exhibit unique dynamic amplification characteristics, as reflected in the values for a_p in Tables 6.3-1 and 6.4-1. Thus, for the calculation of the component integrity and connection to the supporting structure, greater forces are used, as a percentage of component mass, than are typically calculated for the overall seismic-force-resisting system. It is the intent of this provision that component forces be accommodated in the design of the structure as required to prevent local overstress of the immediate vertical and lateral load-carrying systems. Inasmuch as the component masses are included, explicitly or otherwise, in the design of the seismic-force-resisting system, it is generally sufficient for verification of a complete load path to check only for local overstress conditions in the vicinity of the component in question. One approach to achieve this is to check the capacity of the first structural element in the load path (for example, the floor beam directly under a component) for combined dead, live, operating, and seismic loads, using the horizontal and vertical loads from Sec. 6.2.6 for the seismic demand. This procedure is repeated for each structural element or connection in the load path until the load case including horizontal and vertical loads from Sec. 6.2.6 no longer governs the design of the element. This will occur when the component design loads generated by Sec. 6.2.6 become small relative to the dead and live load demands on the structural element. Where component forces have increased due to the nature of the anchorage system, these load increases, which take the form of reductions in R_p, or increase s in F_p, need not be considered in the check of the load path.

An area of concern that is often overlooked is the reinforcement and positive connection of housekeeping slabs to the supporting structure. Lack of such reinforcement and connections has led to costly failures in past earthquakes. Therefore, the housekeeping slabs must be considered as part of the continuous load path be adequately reinforced, and be positively fastened to the supporting structure.

The exact size and location of loads might not be known until the component is ordered. Therefore, the designer should make conservative assumptions in the design of the supporting structural elements. The design of the supporting structural elements must be checked once the final magnitude and location of the design loads have been established.

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 requirements 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 (such as tanks and 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 (such as rubber-in-shear, springs, or air cushions). Heavy mechanical equipment (such as 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 (for example, switchgear and 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 vertical ground motions. Because such frictional resistance cannot be relied upon, positive restraint must be provided for each component.

6.2.6 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 the structure, component anchorage ductility, component importance, component safety hazard upon separation from the structure, structural response, site conditions, and seismic zone. Additional studies have further revised the equation to its present form (Drake and Bachman, 1994 and 1995). In addition, the term C_a has been replaced by the quantity $0.4S_{DS}$ to conform to changes in Chapter 3. BSSC Technical

Subcommittee 8 believes that Eq. 6.2-1, 6.2-3, and 6.2-4 achieve 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 both the structural and component fundamental periods are available. Figure C6.2-1 is from the NCEER work and is an acceptable formulation for a_p as a function of T_p/T. Minor adjustments in the tabulated a_p values were made in the 1997 Edition to be consistent with the 1997 *Uniform Building Code*.

The component response modification factor (R_p) represents the energy absorption capability of the component's structure and attachments. Conceptually, the R_p value considers both the overstrength and deformability of the component's structure and attachments. In the absence of current research, it is believed these separate considerations can be adequately combined into a single factor. The engineering community is encouraged to address the issue and conduct research into the component response modification factor that will advance the state of the art. 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, low deformability element

R_p = 2.5, limited deformability element

R_p = 3.5, high deformability element

Minor adjustments in the tabulated R_p values were made in the 1997 Edition to correlate with F_p values determined in accordance with the 1997 *Uniform Building Code*. Researchers have proposed a procedure for validating values for R_p with respect to documented earthquake performance (Bachman and Drake, 1996).

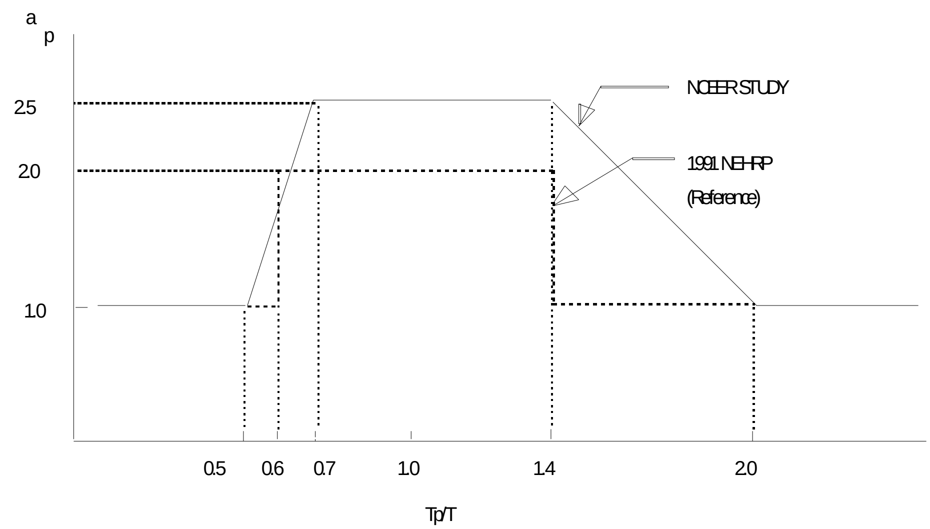


Figure C6.2-1 NCEER formulation for a_p as function of structural and component periods

Eq. 6.2-1 represents a trapezoidal distribution of floor accelerations within the structure, linearly varying from the acceleration at the ground ($0.4S_{DS}$) to the acceleration at the roof ($1.2S_{DS}$). The ground acceleration ($0.4S_{DS}$) is intended to be the same acceleration used as design input for the structure itself and includes site effects.

Examination of recorded in-structure acceleration data in response to large California earthquakes reveals that a reasonable maximum value for the roof acceleration is four times the input ground acceleration to the structure. Earlier work (Drake and Bachman, 1996, 1995 and 1996) indicated that the maximum amplification factor of four seems suitable (Figure C6.2-2). However, a close examination of recently recorded strong motion data at sites with peak ground accelerations in excess of 0.1g indicates that an amplification factor of three is more appropriate (Figure C6.2-3). In the lower portions of the structure (the lowest 20 percent of the structure), both the amplification factors of three and four do not bound the mean plus one standard deviation accelerations. However, the minimum design force in Eq. 6.2-4 provides a lower bound in this region.

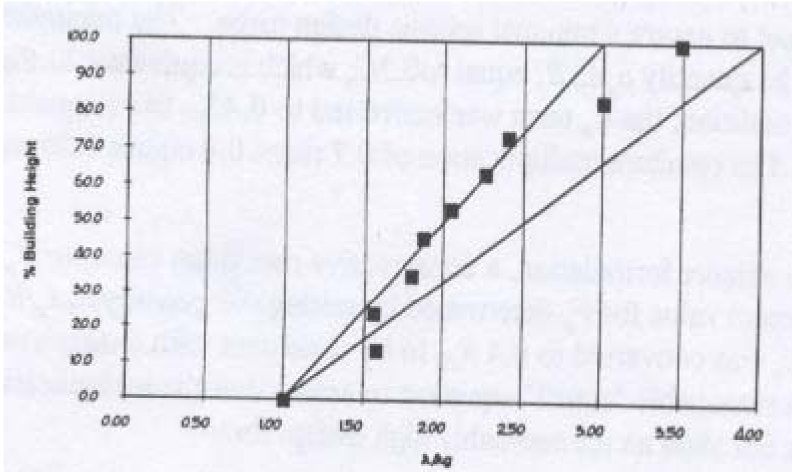


Figure C6.2-2 Revised NEHRP equation vs (Mean + 1σ) acceleration records -all sites

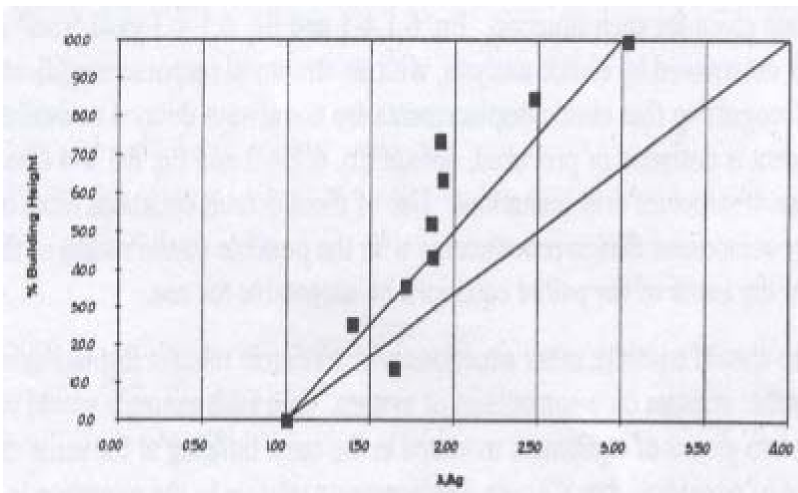


Figure C6.2-3 Revised NEHRP equation vs (Mean +1σ) acceleration records - sites with $A_g \geq 0.1$g

At periods greater than T_s (where $T_s = S_{D1}/S_{DS}$), the acceleration response of structures ground reduces because the design ground motion acceleration response spectra beyond T_s starts to reduce by the ratio of T_s / T (where T is the fundamental period of the primary structure). Since this reduction in the design forces for the primary structure is accounted for in design force equations, it is justifiable to make a similar type of reduction in the design forces of non-structural components. However, in observing the actual in-structure response spectra of acceleration recordings measured at the roof levels of buildings with a range of fundamental periods, this reduction in response typically begins at periods about 25 percent greater than T_S. Therefore, the transition period, T_{flx} at the top of the primary structure at which forces begin reducing by a ratio of T_p/T has been lengthened by 25 percent to account for this observation. At the ground level (z = 0) the adjustment is 0 percent since the effect of the structure response has no influence on the non-structural component response. A linear interpolation is used between the top and bottom of the structure.

A lower limit for F_p is set to assure a minimal seismic design force. The minimum value for F_p was determined by setting the quantity a_pA_p/R_p equal $0.3S_{DS}$ for consistency with current practice.

To meet the need for a simpler formulation, a conservative maximum value for F_p also was set. Eq. 6.2-3 is the maximum value for F_p determined by setting the quantity a_pA_p/R_p equal to 4.0. Eq. 6.2-3 also serves as a reasonable "cutoff" equation to assure that the multiplication of the individual factors does not yield an unreasonably high design force.

To clarify the application of vertical seismic design forces in combination with horizontal design forces and service loads, a cross-reference was provided to Sec. 4.2.2. The value for F_p calculated in accordance with Chapter 6 should be substituted for the value of Q_E in Sec. 4.2.2.

For elements with points of attachment at varying heights, it is recommended that F_p be determined individually at each height (including minima) and the values averaged.

Alternatively for each point of attachment a force F_p shall be determined based on Eq. 6.2-1. Minima and maxima in Sec. 6.2.6 must be utilized in determining each F_p. The weight W_p used in determining each F_p should be based on the tributary weight of the component associated with the point of attachment. For designing the component, the attachment force F_p should be distributed relative to the component's mass distribution over the area used to establish the tributary weight (in the instance of tilt-up walls, a uniform horizontal load would be applied half-way up the wall equal to F_p min.). With the exception of out-of-plane wall anchorage to flexible diaphragms, which is covered by Eq. 4.6-1, each anchorage force should be based on simple statistics determined using all the distributed loads applied to the complete component. Cantilever parapets that are part of a continuous element should be separately checked for parapet forces.

The seismic force on any component must be applied at the center of gravity of the component and must be assumed to act in any horizontal direction. Vertical forces on nonstructural components are specified in Sec. 6.2.6.

6.2.7 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 for use in 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. 6.2-5 and Eq. 6.2-7 produce "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. 6.2-6 and Eq. 6.2-8, which allow the use of structure drift limitations, also are provided. 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 its 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. The height over which D_p, the displacement demand, must be accommodated is often less than the story height h_{sx}, and should be carefully considered. For example, a glazing system sandwiched between two rigid precast concrete spandrel panels may need to accommodate the entire displacement demand in less than 1/3 of the story height. Similar demands can occur when pipes, ducts and conduit that are connected to the top of a tall component are braced to the floor or roof above.

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, avoiding local yielding. Further, it is the intent of the *Provisions* to consider the effects of seismic support relative displacements and displacements caused by seismic forces on mechanical and electrical component assemblies such as piping systems, cable and conduit systems, and other linear systems, and the equipment to which they attach. 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.

6.2.8 Component anchorage. Depending on the specifics of the design condition, ductile design of anchors in concrete or masonry may be intended to satisfy one or all of the following objectives: (1) to ensure adequate load redistribution between anchors in a group, (2) to allow for anchor overload without precipitous failure, and/or (3) to dissipate seismic energy. Unless specific attention is paid to the conditions necessary to ensure the desired hysteretic response (adequate gauge length, anchor spacing, edge distance, steel properties, etc.), it is not recommended that anchors be relied upon for energy dissipation. Inasmuch as the anchor provides the transfer of load from a relatively deformable material (such as steel) to a low deformability material (such as concrete or masonry), achieving deformable, energy-absorbing behavior in the anchor itself is often difficult. On the other hand, the concept of providing a fuse, or deformable link, in the load path to the anchor is encouraged. This approach allows the designer to provide the necessary level of ductility and overstrength in the connection while at the same time protecting the anchor from overload and eliminates the need to balance steel strength and deformability in the anchor with variable edge distances and anchor spacings.

Previous restrictions on the anchor l/d ratio as a means of defining ductile vs. non-ductile anchors have been deleted from the *Provisions* in recognition of the difficulty in defining the conditions necessary for real ductile behavior. For example, a single anchor with the necessary embedment to force ductile failure of the anchor bolt in tension may still experience concrete fracture (a non-ductile failure mode) if the edge distance is small, if the anchor is placed in a group of tension-loaded anchors with reduced spacing, or if the anchor is loaded in shear instead of tension. In fact, many if not most anchor applications, such as building cladding attachments and large equipment anchorages, are subject primarily to shear loading. In these cases, even if the anchor steel is ductile, shear failure of the bolt may be non-ductile, particularly if the deformation of the anchor is constrained by rigid elements on either side of the joint. It is therefore left to the designer to establish the necessary criteria for ductile anchor failure.

Post-installed expansion and undercut anchors may now be qualified as suitable for seismic applications using the testing procedures outlined in ACI 355.2-01, Evaluating the Performance of Post-Installed Mechanical Anchors in Concrete (355.2-01) and Commentary (355.2R-01). The design of qualified anchors in concrete is addressed in Sec. 9.6 of the *Provisions*. No such standard exists as yet for chemical anchors, and caution should be exercised in their use in earthquake environments, particularly with respect to the effects of earthquake-induced cracking of the concrete or masonry on anchor capacity. The capacity of anchors in masonry is rarely governed by steel capacity, and as such masonry anchors should in general be considered to be non-ductile. For this reason, the design of anchors in masonry should be carried out with an R_p of 1.5.

For purposes of the *Provisions*, a chemical anchor is a post-installed anchor rod, usually steel, which is inserted into a drilled hole in concrete or masonry together with a polymer or cementitious grout and which derives its tension capacity primarily from bond. On the other hand, reference to adhesives is intended to include steel plates and other structural elements adhered to the surface of another structural component with adhesive. An example of this type of application is the attachment of computer access floors base plates to a floor slab with epoxy. This type of connection is typically non-ductile.

Allowable loads for anchors should not be increased for earthquake loading. Possible reductions in allowable loads for particular anchor types to account for loss of stiffness and strength should be determined by means of appropriate dynamic testing.

Anchors that are used to support towers, masts, and equipment often are provided with double nuts to allow for leveling during installation. Where baseplate grout is provided at such double-nutted anchors, it should not be relied upon to carry loads since it can shrink and crack or is often omitted altogether. In this case, the anchors are loaded in tension, compression, shear, and flexure and should be designed accordingly. Prying forces on anchors, which result from a lack of rotational stiffness in the connected part, can be critical for anchor design and must be considered explicitly.

For anchorages that are not provided with a mechanism to transfer compression loads, the design for overturning must reflect the actual stiffness of the baseplate, equipment, housing, etc., in determining the location of the compression centroid and the distribution of uplift loads to the anchors.

Possible reductions in allowable loads for particular anchor types to account for loss of stiffness and strength should be determined through appropriate dynamic testing.

While the requirements do not prohibit the use of single anchor connections, it is considered good practice to use at least two anchors in any load-carrying connection whose failure might lead to collapse, partial collapse, or disruption of a critical inertial load path.

Tests have shown that there are consistent shear ductility variations between bolts anchored to drilled or punched plates with nuts and connections using welded, shear studs. Recommendations for design are not presently available but this issue should be considered in critical connections subject to dynamic or seismic loading.

It is important to relate the anchorage demands defined by Chapter 6 with the material capacities defined in the other chapters (e.g., Chapters 9 and 11).

6.2.8.5 Power-actuated fasteners. Generally, power-actuated fasteners in concrete tend to exhibit variations in load capacity that are somewhat larger than post-installed drilled anchors. Therefore, the suitability of power-actuated fasteners should be demonstrated by a simulated seismic test program prior to their use. When properly installed in steel, such fasteners are reliable, showing high capacities with very low variability.

6.2.9 Construction documents. The committee believes that each quality assurance activity specified in Chapter 2 should have a clearly defined basis. As a result, construction documents are required for all components for which Chapter 2 requires special inspection or testing.

The committee believes that, in order to provide a reasonable level of assurance that the construction and installation of components is consistent with the basis of the supporting seismic design, appropriate construction documents are needed. Of particular concern are systems involving multiple trades and suppliers. In these cases, it is important that a registered design professional prepare construction documents for use by the various trades and suppliers in the course of construction.

6.3 ARCHITECTURAL COMPONENTS

The requirements of Sec. 6.3 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 because 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 requirements 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, requirements 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.

6.3.1 Forces and displacements. Components that could be damaged by or could damage other components and are fastened at multiple locations to a structure should be designed to accommodate seismic relative displacements. Such components include glazing, partitions, stairs, and veneers.

Certain types of veneer elements, such as aluminum or vinyl siding and trim, possess high deformability. These systems are generally light and can undergo large deformations without separating from the structure. However, care must be taken when designing these elements to ensure that the low deformability components that may be part of the curtain wall system, such as glazing panels, have been detailed to accommodate the expected deformations without failure.

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 poses a low hazard to life safety. 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 requirements 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 structures, such as those with shear walls.

Differential vertical movement between horizontal cantilevers in adjacent stories (such as cantilevered floor slabs) has occurred in past earthquakes. The possibility of such effects should be considered in the design of exterior walls.

6.3.2 Exterior nonstructural wall elements and connections. The *Provisions* requires that nonbearing wall panels that are attached to or enclose the structure be designed to resist the (inertial) forces and to accommodate movements of the structure resulting from lateral forces or temperature change. The force requirements often overshadow the importance of allowing thermal movement and may therefore require special detailing in order to prevent moisture penetration and allow thermal movements.

Connections should be designed so as to prevent the loss of load-carrying capacity in the event of significant yielding. Between points of connection, panels should be separated from the structure sufficiently to avoid contact due to seismic action.

The *Provisions* requires allowance for story drift. This required allowance can amount to 2 in. (50 mm) or more from one floor to the next and may present a greater challenge to the designer than requirements for the forces. In practice, separations between adjacent panels, intended to limit contact and resulting panel mis-alignment and/or damage under all but extreme building response, are limited to about 3/4 in.

(19 mm) for practical joint detailing with acceptable appearance. The *Provisions* calls for a minimum separation of 1/2 in. (13 mm). The design should be consistent with the manufacturing and construction tolerances of the materials used to achieve this dimension.

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. Appropriate isolation can be achieved by means of slots, but the use of long rods that flex is preferable because this approach does not depend on precise installation to achieve the desired action. The rods must be designed to carry tension and compression in addition to induced flexural stresses. Care must be used in allowing inelastic bending in the rods. Threaded rods pushed into the strain-hardening region of the stress-strain curve are subject to brittle low-cycle fatigue failures. Floor-to-floor wall panels are usually rigidly attached to and move with the floor structure nearest the panel bottom. In this condition, isolation connections are used at the upper attachments so that panels translate with the load supporting structure below and are not subjected to large in-plane forces due to movement of the building. Panels also can be supported at the top with isolation connections 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.

To minimize the effects of thermal movements and shrinkage on architectural cladding panels, the connection system is generally detailed to be statically determinate. As a result, cladding panel support systems often lack redundancy and failure of a single connection can have catastrophic consequences. In recognition of this, the *Provisions* require that fasteners be designed for approximately 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 of 4 has been incorporated into the a_p and R_p factors in consideration of installation and material variability and the consequences of a brittle connection failure in a statically determinate system.

6.3.3 Out-of-plane bending. Most walls are subject to out-of-plane forces when a building is shaken by 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 or materials with low flexural strength. The conventional limits based upon deflections as a proportion of the span may be used with the applied force as derived in Sec. 6.2.6.

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 undergo 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.

6.3.4 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 earthquakes in California has raised two concerns:

1. Support of the individual panels at walls and expansion joints, and
2. Interaction with fire sprinkler systems.

In an attempt to address these concerns, alternate methods were developed in a cooperative effort by representatives of the ceiling and fire sprinkler industries and registered design professionals. It is hoped that future research and investigation will result in further improvements in the *Provisions*.

Consideration must be given to the placement of seismic bracing, 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 interaction of components.

Dynamic testing of suspended ceiling systems constructed according to the requirements of current industry seismic standards (*UBC* Standard 25-2) performed by ANCO Engineers, Inc. (1983) has demonstrated that the splayed wires, even with the vertical compression struts, 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 sufficient clearance is provided at penetrations and closure angles are wide enough.

The lateral seismic restraint for a non-rigidly braced suspended ceiling is primarily provided by the ceiling coming into 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.

6.3.5 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 may 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.

For systems with floor stringers, it is acceptable practice 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 non-conservative to use the design vertical load simultaneously with the design seismic force when considering anchor bolts, pedestal bending, and pedestal welds to base plates. The maximum concurrent vertical load when considering overturning effects is therefore limited to the value of 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, 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.

6.3.6 Partitions. Partitions are sometimes designed to run only from floor to a suspended ceiling which provides doubtful lateral support. Partitions subject to these requirements 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.

6.3.7 Glass in glazed curtain walls, glazed storefronts, and glazed partitions. Glass performance in earthquakes can fall into one of four categories:

1. The glass remains unbroken in its frame or anchorage.
2. The glass cracks but remains in its frame or anchorage while continuing to provide a weather barrier, and to be otherwise serviceable.
3. The glass shatters but remains in its frame or anchorage in a precarious condition, likely to fall out at any time.
4. The glass falls out of its frame or anchorage, either in fragments, shards, or whole panels.

Categories 1. and 2. provide both life safety and immediate occupancy levels of performance. In the case of category 2., even though the glass is cracked, it continues to provide a weather enclosure and barrier, and its replacement can be planned over a period of time. (Such glass replacement need not be performed in the immediate aftermath of the earthquake.) Categories 3. and 4. cannot provide for immediate occupancy, and their provision of a life safety level of performance depends on the post-breakage characteristics of the glass and the height from which it can fall. Tempered glass shatters into multiple, pebble-size fragments that fall from the frame or anchorage in clusters. These broken glass clusters are relatively harmless to humans when they fall from limited heights, but when they fall from greater heights they could be harmful.

The requirement that $\Delta_{fallout}$ not be less than $1.25ID_P$ is derived from *Earthquake Safety Design of Windows*, published in November 1982 by the Sheet Glass Association of Japan. Eq. 6.3-1 is based on a similar equation in Bouwkamp and Meehan (1960) that permits calculation of the story drift required to cause glass-to-frame contact in a given rectangular window frame. Both calculations are based on the principle that a rectangular window frame (specifically, one that is anchored mechanically to adjacent stories of the primary structural system of the building) becomes a parallelogram as a result of story drift, and that glass-to-frame contact occurs when the length of the shorter diagonal of the parallelogram is equal to the diagonal of the glass panel itself.

The 1.25 factor in the requirements described above reflect uncertainties associated with calculated inelastic seismic displacements in building structures. Wright (1989) stated that “post-elastic deformations, calculated using the structural analysis process, may well underestimate the actual building deformation by up to 30 percent. It would therefore be reasonable to require the curtain wall glazing system to withstand 1.25 times the computed maximum interstory displacement to verify adequate performance.” Therefore, Wright’s comments form the basis for employing the 1.25 factor in these requirements.

Introduction. Seismic design requirements for glass in building codes have traditionally been non-existent or have been limited to the general statement that “drift be accommodated.” No distinction has been made regarding the seismic performance of different types of glass, different frames, and different glazing systems. Yet, significant differences exist in the performance of various glass types subjected to simulated earthquake conditions. Controlled laboratory studies were conducted to investigate the cracking resistance and fallout resistance of different types of glass installed in the same storefront and mid-rise wall systems. Effects of glass surface prestress, lamination, wall system type, and dry versus structural silicone glazing were considered. Laboratory results revealed that distinct magnitudes of story drift cause glass cracking and glass fallout in each glass type tested. Notable differences in seismic resistance exist between glass types commonly used in contemporary building design.

Test rig and experimental plan. In-plane dynamic racking tests were performed using the rig shown in Figure C6.3-1. Rectangular steel tubes at the top and bottom of the facility are supported on roller assemblies, which permit only horizontal motion of the tubes. The bottom steel tube is driven by a computer-controlled hydraulic ram, while the top tube is attached to the bottom tube by means of a fulcrum and pivot arm assembly. This mechanism causes the upper steel tube to displace the same amount as the lower steel tube, but in the opposite direction, which doubles the amount of story drift that can be imposed on a test specimen from ± 76 mm (± 3 in.) to ± 152 mm (± 6 in.). The test facility accommodated up to three glass test panels, each 1.5 m (5 ft) wide by 1.8 m (6 ft) high. A more detailed description of the dynamic racking test rig is included in Behr and Belarbi (1996).

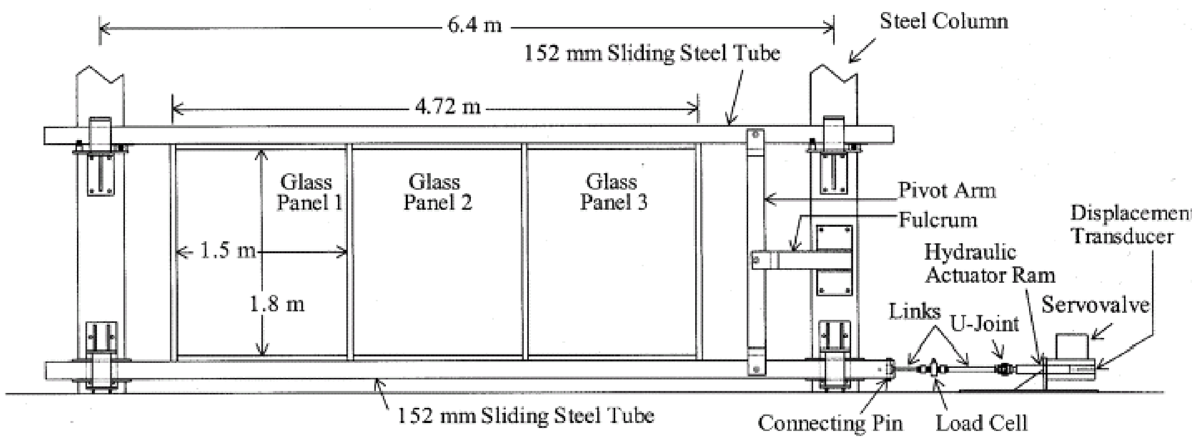


Figure C6.3-1 Dynamic racking test rig.

Several types of glass, shown in Table C6.3-1, were tested under simulated seismic conditions in the storefront and mid-rise dynamic racking tests. These glass types, along with the wall systems employed in the tests, were selected after polling industry practitioners and wall system designers for their opinions regarding common glass types and common wall system types employed in contemporary storefront and mid-rise wall constructions.

Storefront wall system tests. Tests were conducted on various glass types that were dry-glazed within a wall system, as commonly used in storefront applications. Loading histories for the storefront wall system tests were based on dynamic analyses performed on a "typical" storefront building that was not designed specifically for seismic resistance (Pantelides et al., 1996). Two types of tests were conducted on the storefront wall systems: (1) serviceability tests, wherein the drift loading history of the glass simulated the response of a storefront building structure to a "maximum probable" earthquake event; and (2) ultimate tests, wherein drift amplitudes were twice those of the serviceability tests, which was a simplified means of approximating the loading history of a "maximum credible" earthquake event. As indicated in Table C6.3-1, five glass types were tested, all dry-glazed in a storefront wall system. Three glass panels were mounted side by side in the test facility, after which horizontal (in-plane) racking motions were applied.

Table C6.3-1 Glass Types Included in Storefront and Mid-Rise Dynamic Racking Tests

Glass Type	Storefront Tests	Mid-Rise Tests
6 mm (1/4 in.) annealed monolithic	✓	✓
6 mm (1/4 in.) heat-strengthened monolithic		✓
6 mm (1/4 in.) fully tempered monolithic	✓	✓

6 mm (1/4 in.) annealed monolithic with 0.1 mm pet film (film not anchored to wall system frame)		✓
6 mm (1/4 in.) annealed laminated	✓	✓
6 mm (1/4 in.) heat-strengthened laminated		✓
6 mm (1/4 in.) heat-strengthened monolithic spandrel		✓
25 mm (1 in.) annealed insulating glass units	✓	✓
25 mm (1 in.) heat-strengthened insulating glass units		✓

The serviceability test lasted approximately 55 seconds and incorporated drift amplitudes ranging from ± 6 to ± 44 mm (± 0.25 to ± 1.75 in.). The drift pattern in the ultimate test was formed by doubling each drift amplitude in the serviceability test. Both tests were performed at a nominal frequency of 0.8 Hz.

Experimental results indicated that for all glass types tested, serviceability limit states associated with glass edge damage and gasket seal degradation in the storefront wall system were exceeded during the moderate earthquake simulation (that is, the serviceability test). Ultimate limit states associated with major cracking and glass fallout were reached for the most common storefront glass type, 6 mm (1/4 in.) annealed monolithic glass, during the severe earthquake simulation (that is, the ultimate test). This observation is consistent with a reconnaissance report of damage resulting from the Northridge Earthquake (EERI, 1994). More information regarding the storefront wall system tests is included in Behr, Belarbi, and Brown (1995). In addition to the serviceability and ultimate tests, increasing-amplitude "crescendo tests," similar to those described below for the mid-rise tests, were performed at a frequency of 0.8 Hz on selected storefront glass types. Results of these crescendo tests are reported in Behr, Belarbi, and Brown (1995) and are included in some of the comparisons made below.

Mid-rise curtain wall system tests. Another series of tests focused on the behavior of glass panels in a popular curtain wall system for mid-rise buildings. All mid-rise glass types in Table C6.3-1 were tested with a dry-glazed wall system that uses polymeric (rubber) gaskets wedged between the glass edges and the curtain wall frame to secure each glass panel perimeter. In addition, three glass types were tested with a bead of structural silicone sealant on the vertical glass edges and dry glazing gaskets on the horizontal edges (that is, a "two-side structural silicone glazing system"). Six specimens of each glass type were tested.

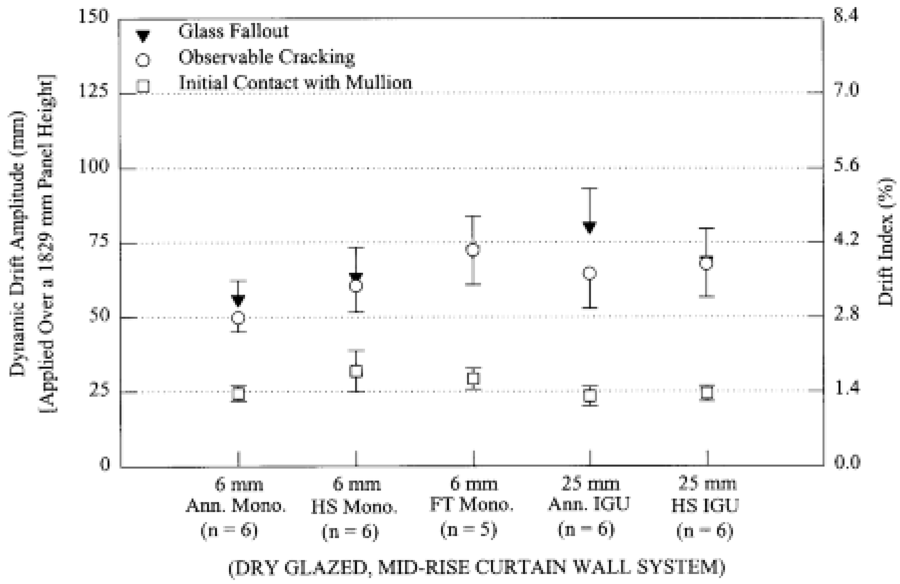
Crescendo tests were performed on all mid-rise test specimens. As described by Behr and Belarbi (1996), the crescendo test consisted of a series of alternating "ramp-up" and "constant amplitude" intervals, each containing four sinusoidal-shaped drift cycles. Each drift amplitude "step" (that is, the increase in amplitude between adjacent constant amplitude intervals, which was achieved by completing the four cycles in the intermediary ramp-up interval) was ± 6 mm (± 0.25 in.). The entire crescendo test sequence lasted approximately 230 seconds. Crescendo tests on mid-rise glass specimens were conducted at 1.0 Hz for dynamic racking amplitudes from 0 to 114 mm (0 to 4.5 in.), 0.8 Hz for amplitudes from 114 to 140 mm (4.5 to 5.5 in.), and 0.5 Hz for amplitudes from 140 to 152 mm (5.5 to 6 in.). These frequency reductions at higher racking amplitudes were necessary to avoid exceeding the capacity of the hydraulic actuator ram in the dynamic racking test rig.

The drift magnitude at which glass cracking was first observed was called the "serviceability drift limit," which corresponds to the drift magnitude at which glass damage would necessitate glass replacement. The drift magnitude at which glass fallout occurred was called the "ultimate drift limit," which corresponds to the drift magnitude at which glass damage would become a life safety hazard. This ultimate drift limit for architectural glass is related to "$\Delta_{fallout}$" in Sec. 6.3.7 of the *Provisions*, noting that horizontal racking displacements (drifts) in the crescendo tests were typically applied to test specimens having panel heights of only 1.8 m (6 ft).

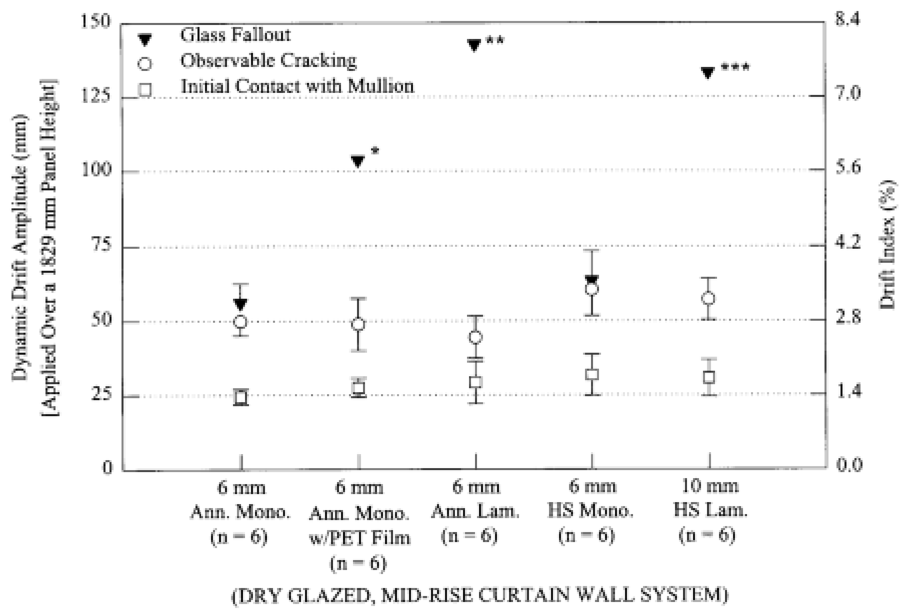
In addition to recording the serviceability drift limit and ultimate drift limit for each glass test specimen, the drift magnitude causing first contact between the glass panel and the aluminum frame was also recorded. To establish when this contact occurred, thin copper wires were attached to each corner of the glass panel and were connected to an electronics box. If the copper wire came into contact with the aluminum frame, an indicator light on an electronics box was actuated. Measured drifts causing glass-to-aluminum contact correlated well with those predicted by Eq. 6.3-1.

Glass failure patterns from crescendo tests. Glass failure patterns were recorded during each storefront test and mid-rise test. Annealed monolithic glass tended to fracture into sizable shards, which then fell from the curtain wall frame. Heat-strengthened monolithic glass generally broke into smaller shards than annealed monolithic glass, with the average shard size being inversely proportional to the magnitude of surface compressive prestress in the glass. Fully tempered monolithic glass shattered into much smaller, cube-shaped fragments. Annealed monolithic glass with unanchored 0.1 mm (4 mil) PET film also fractured into large shards, much like un-filmed annealed monolithic glass, but the shards adhered to the film. However, when the weight of the glass shards became excessive, the entire shard/film conglomeration sometimes fell from the glazing pocket as a unit. Thus, unanchored 0.1 mm PET film was not observed to be totally effective in terms of preventing glass fallout under simulated seismic loadings, which agrees with field observations made in the aftermath of the 1994 Northridge Earthquake (Gates and McGavin, 1998). Annealed and heat-strengthened laminated glass units experienced fracture on each glass ply separately, which permitted these laminated glass units to retain sufficient rigidity to remain in the glazing pocket after one (or even both), glass plies had fractured due to glass-to-aluminum contacts. Annealed and heat-strengthened laminated glass units exhibited the highest resistance to glass fallout during the dynamic racking tests.

Quantitative drift limit data from crescendo tests. Serviceability and ultimate drift limit data obtained during the crescendo tests are presented in four panels in Figure C6.3-2. Figure C6.3-2a shows the effects of glass surface prestress (that is, annealed, heat-strengthened and fully tempered glass) on seismic drift limits; Figure C6.3-2b shows the effects of lamination (that is, monolithic glass, monolithic glass with unanchored 0.1 mm PET film, and laminated glass); Figure C6.-2c shows the effects of wall system type (that is, lighter, more flexible, storefront wall system versus the same glass types tested in a heavier, stiffer, mid-rise wall system); and Figure C6.3-2d shows the effects of structural silicone glazing (that is, dry glazing versus two-side structural silicone glazing). Each symbol plotted in Figure C6.3-2 is the mean value for specimens of a given glass type, along with ± one standard deviation error bars. In those cases where error bars for a particular glass type overlap, only one side of the error bar is plotted. In cases where the glass panel did not experience fallout by the end of the crescendo test, a conservative ultimate drift limit magnitude of 152 mm (6 in.) (the racking limit of the test facility) is assigned for plotting purposes in Figure C6.3-2. (This ultimate drift limit, shown with a "▾" symbol in Figure C6.3-2, is related to the term "$\Delta_{fallout}$" in Sec. 6.3.7 of the *Provisions*.) No error bars are plotted for these "pseudo data points," since the drift magnitude at which the glass panel would actually have experienced fallout could not be observed; certainly, the actual ultimate drift limits for these specimens are greater than ±152 mm (± 6 in.).



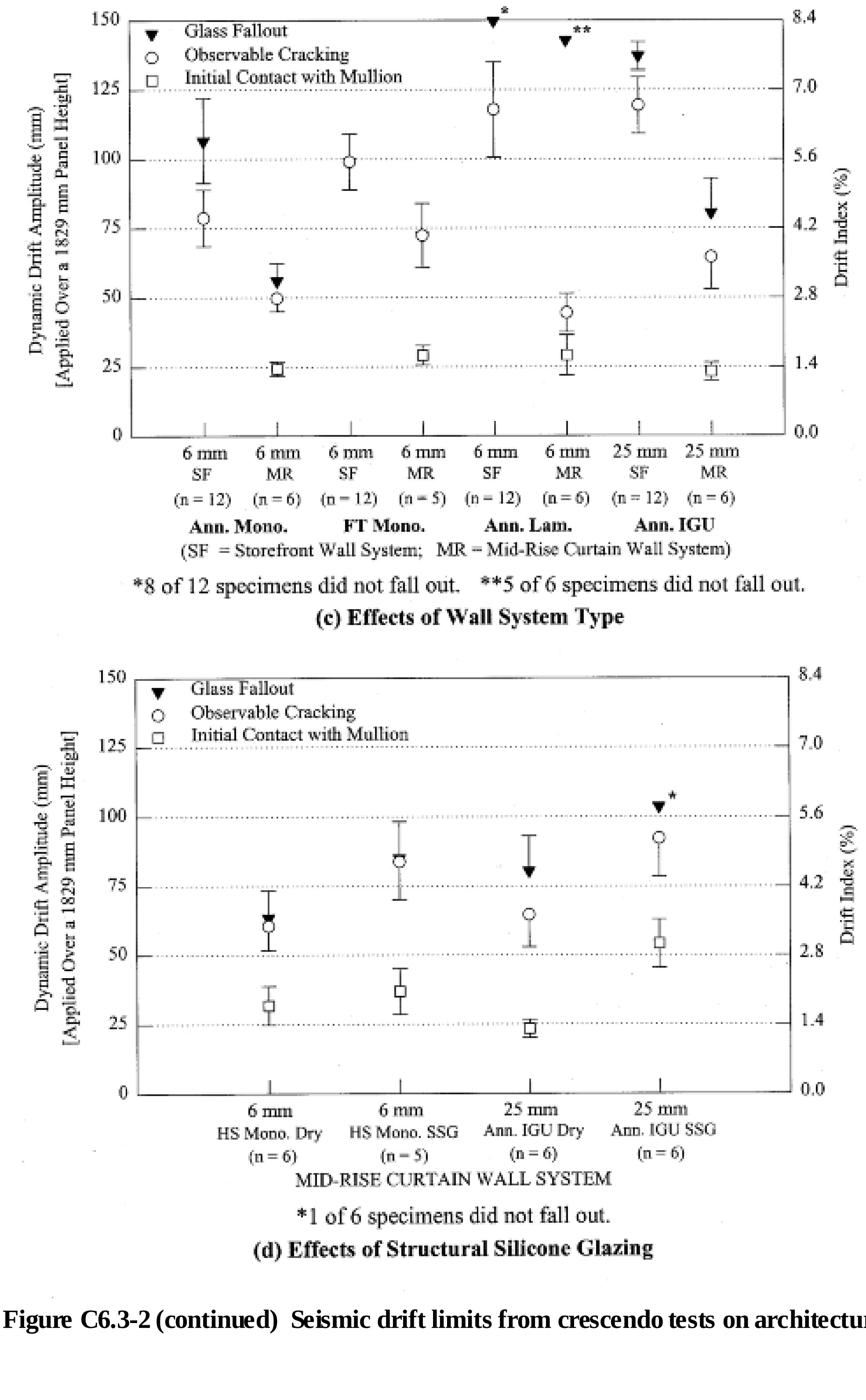
(a) Effects of Glass Surface Prestress



*1of 6 specimens did not fall out. **5 of 6 specimens did not fall out. ***2 of 6 specimens did not fall out.

(b) Effects of Lamination

Figure C6.3-2 Seismic drift limits from crescendo tests on architectural glass



(c) Effects of Wall System Type

(d) Effects of Structural Silicone Glazing

Figure C6.3-2 (continued) Seismic drift limits from crescendo tests on architectural glass

The ±152 mm (± 6 in.) racking limit of the test rig, when applied over the 1829 mm (72 in.) height of glazing panel specimens represents a severe story drift index of more than 8 percent. This 8 percent drift index exceeds, by a significant margin, provisions in Sec. 4.5.1 (Table 4.5-1) that set allowable drift limits between 0.7 percent and 2.5 percent, depending on structure type and Seismic Use Group. Thus, the drift limits, Δ_a, in Table 4.5-1 are considerably lower than the racking limits of the laboratory facility used for the crescendo tests. In building design, however, values of $\Delta_{fallout}$ would need to be significantly higher than the story drifts exhibited by the primary building structure in order to provide an acceptable safety margin against glass fallout.

Summary observations from Figure C6.3-2. Effects of glass surface prestress - Figure C6.3-2a illustrates the effects of glass surface prestress on observed seismic drift limits. To eliminate all variables except for glass surface prestress, data from only the mid-rise curtain wall tests are plotted. Slight increases in cracking and fallout drift limits can be seen for 6 mm (0.25 in.) monolithic glass as the level of glass surface prestress increases from annealed to heat-strengthened to fully tempered glass. However, effects of glass surface prestress on observed seismic drift limits were statistically significant only when comparing 6 mm fully tempered monolithic glass to 6 mm annealed monolithic glass. All six of the 6 mm fully tempered monolithic glass specimens shattered when initial cracking occurred, causing the entire glass panels to fall out. Similar behavior was observed in four of the six 6 mm heat-strengthened monolithic glass specimens. No appreciable differences in seismic drift limits existed between annealed and heat-strengthened 25 mm (1 in.) insulating glass units.

Effects of lamination -- Figure C6.3-2b shows the effects of lamination configuration on seismic drift limits. Lamination had no appreciable effect on the drift magnitudes associated with first observable glass cracking. In a dry-glazed system, the base glass type (and not the lamination configuration) appeared to control the drift magnitude associated with glass cracking. However, lamination configuration had a pronounced effect on glass fallout resistance (that is, $\Delta_{fallout}$). Specifically, monolithic glass types were more prone to glass fallout than were either annealed monolithic glass with unanchored 0.1 mm PET film or annealed laminated glass. All six annealed monolithic glass panels experienced glass fallout during the tests; five of six annealed monolithic glass specimens with unanchored 0.1 mm PET film experienced fallout; only one of six annealed laminated glass panels experienced fallout.

Laboratory tests also showed that heat-strengthened laminated glass had higher fallout resistance than did heat-strengthened monolithic glass. Heat-strengthened monolithic glass panels fell out at significantly lower drift magnitudes than did heat-strengthened laminated glass units. Heat-strengthened laminated glass units tended to fall out in one large piece, instead of exhibiting the smaller shard fallout behavior of heat-strengthened monolithic glass.

Effects of wall system type -- Figure C6.3-2c illustrates the effects of wall system type on observed seismic drift limits. For all four glass types tested in both the storefront and mid-rise wall systems, the lighter, more flexible storefront frames allowed larger drift magnitudes before glass cracking or glass fallout than did the heavier, stiffer mid-rise curtain wall frames. This observation held true for all glass types tested in both wall system types.

Effects of structural silicone glazing -- As shown in Figure C6.3-2d, use of a two-side structural silicone glazing system increased the dynamic drift magnitudes associated with first observable glass cracking in both heat-strengthened monolithic glass and annealed insulating glass units. During the crescendo tests, glass panels were observed to “walk” horizontally across the frame after the beads of structural silicone sealant had sheared. Because the mid-rise curtain wall crescendo tests were performed on single glass panels, the glass specimen was unobstructed as it walked horizontally across the frame. In a multi-panel curtain wall assembly on an actual building, adjacent glass panels could collide, which could induce glass cracking at lower drift magnitudes than those observed in the single-panel tests performed in this study. It is also clear from Figure C6.3-2d that glass specimens with two-side structural silicone glazing exhibited higher resistance to glass fallout than did comparable dry-glazed glass specimens.

Conclusion. Dynamic racking tests showed that distinct and repeatable dynamic drift magnitudes were associated with glass cracking and glass fallout in various types of glass tested in storefront and mid-rise wall systems. Seismic resistance varied widely between glass types commonly employed in contemporary building design. Annealed and heat-strengthened laminated glass types exhibited higher resistance to glass fallout than did monolithic glass types. Annealed monolithic glass with unanchored 0.1 mm PET film exhibited total fallout of the glass shard/adhesive film conglomeration in five out of six of the crescendo tests performed.

Glass panels glazed within stiffer aluminum frames were less tolerant of glass-to-aluminum collisions and were associated with glass fallout events at lower drift magnitudes than were the same glass types tested in a more flexible aluminum frame. Glazing details were also found to have significant effects on the seismic performance of architectural glass. Specifically, architectural glass within a wall system using a structural silicone glaze on two sides exhibited higher seismic resistance than did identical glass specimens dry-glazed on all four sides within a comparable wall framing system.

6.4 MECHANICAL AND ELECTRICAL COMPONENTS

The primary focus of these requirements is on the design of attachments and equipment supports for mechanical and electrical components.

The requirements are intended to reduce the hazard to life posed by the loss of component structural stability or integrity. The requirements should increase the reliability of component operation but do not directly address the assurance of functionality. For critical components where operability is vital, the requirements of Sec. 2.4.5 provide methods for seismically qualifying the component.

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 requirements is only to design the support and the means by which the component is attached to the structure, defined in the Definitions as “supports” and “attachments.” For the second safety level, failure of the mechanical or electrical equipment itself poses a significant hazard. This could be a case either of failure of a containment having hazardous contents or contents required after the earthquake or of functional failure of a component required to remain operable after an earthquake. In this second category, the intent of these requirements 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 requirements 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, but the hazard of a large fluid spill associated with seismic failure of large, flat-bottom tanks suggests that the design of many, if not most, of such tanks should consider earthquake loads. Distinguishing between large and small, in this case, may require an assessment of potential damage caused by a spill of the fluid contents as outlined in Sec. 6.2.3 and Chapter 1 of ASCE-7

It is intended that the requirements 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 directly affects only structural integrity and stability. Function and operability of mechanical and electrical components may be affected only indirectly 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 reasonable to conclude that if structural integrity and stability are maintained, function and operability after an earthquake will be provided for many 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 leaktightness is re-established after the earthquake. Judgment may suggest a more conservative design related in some manner to the perceived hazard than would otherwise be provided by these requirements.

It is not intended that all equipment or parts of equipment be designed for seismic forces. Determination of whether these requirements 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. When I_p is specified to be 1.0, damage to, or even failure of, a piece or part of a component is not a concern of these requirements 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 requirements even though the component itself may suffer damage. Judgment will be required if the intent of these requirements 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 requirements 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. Certain suspended components are exempt from lateral bracing requirements, provided they meet prescriptive force and interaction requirements.

It is not the intent of the *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. Where 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 provides such equipment with 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 an assessment of equipment ruggedness will b used in determining an appropriate method and extent of 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 requirements that such codes and standards having earthquake design guidance be used; normally the developers of such codes and standards are more familiar with the expected failure modes of the components for which they have developed design and construction rules. In particular, such codes and standards may be based on considerations that are not immediately obvious to the structural design professional. For example, in the design of industrial piping, seismic loads are not typically additive to thermal expansion loads. Given the potential for misunderstanding and mis-application of codes and standards specific to the design of mechanical and electrical systems, it is recommended that a registered design professional familiar with these *Provisions*, as well as the those of the referenced code or standard used to evaluate the capacity of the mechanical or electrical components, should be involved in the review and acceptance of the seismic design process for such components. 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 (especially if constructed of ductile materials) will not break up or break away from its supports in such a way as to pose a life-safety hazard. Earthquake damage surveys confirm this.

Specific guidance for selected components or conditions is provided in Sec. 6.4.5 through 6.4.9.

Testing is a well established alternative method of seismic qualification for small to medium size equipment. Several national standards have testing requirements adaptable for seismic qualification. AC 156, *Acceptance Criteria for Seismic Qualification Testing of Nonstructural Components*, is an acceptable shake table testing protocol, which meets the force requirements of the *Provisions* as well as ASCE 7-02.

6.4.1 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 (that is, 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 to excite the mode of vibration with the highest fundamental period by in-situ tests, relatively significant input levels of motion are required (that is, the flexibility of the base and attachment needs to be exercised). A procedure such as the resonant frequency search in AC 156 may used to identify the dominant modes of vibration of the component.

Many types of mechanical 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, 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.

6.4.2 and 6.4.3 Mechanical and electrical components. Past earthquakes have demonstrated that most mechanical and electrical equipment is inherently rugged and performs well provided that it is properly attached to the structure. This is because the operational and transportation loads for which the equipment is designed are typically larger than those due to earthquakes. For this reason, the requirements focus primarily on equipment anchorage and attachments. However, it was felt that mechanical components required to maintain containment of flammable or hazardous materials should themselves be designed for seismic forces.

In addition, the reliability of equipment operability after an earthquake can be increased if the following items are also considered in design:

1. Internal assemblies, subassemblies, and electrical contacts are attached sufficiently to prevent their being subjected to differential movement or impact with other internal assemblies or the equipment enclosure.
2. Operators, motors, generators, and other such components that are functionally attached to 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. Any ceramic or other nonductile components in the seismic load path are specifically evaluated.
4. Adjacent electrical cabinets are bolted together and cabinet lineups are prevented from banging into adjacent structural members.

Components that could be damaged or could damage other components and are fastened to multiple locations of a structure should be designed to accommodate seismic relative displacements. Examples of components that should be designed to accommodate seismic relative displacements include bus ducts, cable trays, conduit, elevator guide rails, and piping systems.

6.4.4 Supports and 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 support conditions. However, components made of less ductile materials can only accommodate relative displacement effects if appropriate flexibility or flexible connections are provided.

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 less prone to damage and are less likely to fracture and fall provided the supports can accommodate the imposed loads.

It is the intent of these requirements to ensure that all mechanical and electrical component supports 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.

6.4.5 Utility and service lines. For essential facilities, auxiliary on-site mechanical and electrical utility sources are recommended. It is recommended that an appropriate clause be included if existing codes for the jurisdiction do not presently provide for it.

Sec. 6.4.5 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. 4.3.1 and should be in accordance with *Provisions* Sec. 6.2.7.

Consideration may be necessary for nonessential piping that carries quantities of materials that could damage essential utilities in the event of pipe rupture.

Following a review of information from the Northridge and Loma Prieta earthquakes and discussions with gas company personnel, automatic earthquake shutoff of gas lines at structure entry points is no longer required. The primary justification for this is the consensus opinion that shutoff devices tend to cause more problems than they solve. Commercially available shutoff devices are often susceptible to inadvertent shutoff caused by passing vehicles and other non-seismic vibrations. This leads to disruption of service and often requires that local gas companies reset such devices and relight any pilot lights. In an earthquake, the majority of shutoff devices which actuate will be attached to undamaged gas lines. This results in a huge relight effort for the local utility at a time when resources are typically at a premium. If the earthquake occurs during the winter, a greater life hazard may exist from a lack of gas supply than from potential gas leaks. In the future, as shutoff devices improve and gas-fired appliances which use pilots are phased out, it may be justified to require shutoff devices.

This is not meant to discourage individuals and companies from installing shutoff devices. In particular, individuals and companies who are capable of relighting gas-fired equipment should seriously consider installation of these devices. In addition, gas valves should be closed whenever leaks are detected.

6.4.6 HVAC ductwork. Experience in past earthquakes has shown that, in general, HVAC duct systems are rugged and perform well in strong shaking motions. Bracing in accordance with the Sheet Metal and Air Conditioning Contractors National Association SMACNA 80, SMACNA 95, and SMACNA 98 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. Therefore, 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 large amplitude cycles of bending stress, should be avoided.

Some ductwork systems carry hazardous materials or must remain operational during and after an earthquake. Such ductwork system would be assigned a value of I_p greater than 1.0. A detailed engineering analysis for these systems should be performed.

All equipment attached to the ducts and weighing more than 75 lb (334 N) , such as fans, humidifiers, and heat exchangers, should be braced independently of the duct. Unbraced in-line equipment can damage the duct by swinging and impacting it during an earthquake. Items attached to the duct (such as dampers, louvers, and air diffusers) 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 references listed in Sec. 6.1.1 may be used.

6.4.7 Piping systems. Experience in past earthquakes has shown that, in general, piping systems are rugged and perform well in strong shaking motions. Numerous standards and guidelines have been developed covering a wide variety of piping systems and materials. Construction in accordance with current requirements of the referenced national standards 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 standards be used to design piping systems provided that the force and displacement demands are equal to or exceed those outlined in Sec. 6.2.6 and 6.2.7 and provision is made to mitigate seismic interaction issues not normally addressed in the national standards.

The following industry standards, while not adopted by ANSI, are in common use and may be appropriate reference documents for use in the seismic design of piping systems: SMACNA *Guidelines for the Seismic Restraint of Mechanical Systems* and ASHRAE CH 50-95 *Seismic Restraint Design Piping*.

6.4.8 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 requirements of the ASME *Boiler and Pressure Vessel Code* (ASME BPV) 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 demands are equal to or exceed those outlined in Sec. 6.2.6 and 6.2.7. Until such nationally recognized codes incorporate force and displacement requirements comparable to the requirements of Sec. 6.2.6 and 6.2.7, it is nonetheless the intention to use the design acceptance criteria and construction practices of those codes.

6.4.9 Elevators. The ASME *Safety Code for Elevators and Escalators* (ASME A17.1) has adopted many requirements 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*.

6.4.9.2 Elevator machinery and controller supports and attachments. ASME A17.1 has no seismic requirements 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*.

6.4.9.3 Seismic switches. The purpose of seismic switches as used here is different from that of ASME A17.1, which has incorporated several requirements to improve the seismic response of elevators (such as rope snag point guard, rope retainer guards, and guide rail brackets) and which does 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 defining for 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.

6.4.9.4 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 the *Provisions* (for example, resilient mounting components or certain protecting devices) require maintenance to ensure their reliability and to provide 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 has not been determined, so requirements to cover these situations have not been included.

Tenant improvements. It is intended that the requirements in Chapter 6 also apply to newly constructed tenant improvements that are listed in Tables 6.3-1 and 6.4-1 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 raised here because it is believed that all members of the profession should clearly understand that Chapter 6 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 considered in the structural engineer’s approach to seismic safety if the design profession as a whole is to make any meaningful impact on the public awareness of this matter. It is hoped that, as the design profession gains more knowledge and sophistication in the use of seismic design, a more comprehensive approach to earthquake design requirements will be developed.

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End note: The American Architectural Manufacturers Association (AAMA) has issued AAMA 501.4-2000: "Recommended Static Test Method for Evaluating Curtain Wall and Storefront Systems Subjected to Seismic and Wind Induced Interstory Drifts." In contrast with the dynamic displacements employed in the crescendo tests described in this section, static displacements are employed in AAMA's recommended test method. Correlations between the results of the static and dynamic test methods have not yet been established with regard to the seismic performance of architectural glazing systems.

Note that this section addresses glass in frames subject to interstory drift. Glass frames not subject to interstory drift, such as freestanding guards and screens, are not covered by this section.

Chapter 7 Commentary

FOUNDATION DESIGN REQUIREMENTS

7.1 GENERAL

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

7.2 GENERAL DESIGN REQUIREMENTS

7.2.2 Soil capacities. This section requires that the building foundation without seismic forces applied must be adequate to support the building gravity load. When seismic effects are considered, the soil capacities can be increased considering the short time of loading and the dynamic properties of the soil. It is noted that the Appendix to Chapter 7 introduces into the *Provisions* ultimate strength design (USD) procedures for the geotechnical design of foundations. The *Commentary* Appendix to Chapter 7 provides additional guidance and discussion of the USD procedures.

7.2.3 Foundation load-deformation characteristics. The Appendix to Chapter 7 (*Provisions* and *Commentary* (Sec. A7.2.3) provides guidance on modeling load-deformation characteristics of the foundation-soil system (foundation stiffness). The guidance contained therein covers both linear and nonlinear analysis methods.

7.3 SEISMIC DESIGN CATEGORY B

There are no special seismic provisions for the design of foundations for buildings assigned to Seismic Design Category B.

7.4 SEISMIC DESIGN CATEGORY C

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

7.4.1 Investigation. This section reviews procedures that are commonly used for evaluating potential site geologic hazards due to earthquakes, including slope instability, liquefaction, and surface fault rupture. Geologic hazards evaluations should be carried out by qualified geotechnical professionals and documented in a written report.

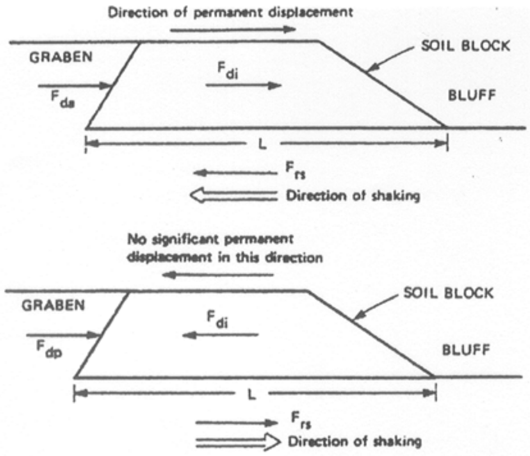
Screening Evaluation. Evaluation of geologic hazard may initially consist of a screening evaluation. If the screening evaluation clearly demonstrates that a hazard is not present, then more detailed evaluations, using procedures such as those described in the following sections, need not be conducted. Reference to the following publications are suggested for guidelines on screening evaluations: California Division of Mines and Geology (1997) – slope instability; Blake et al. (2002) and Stewart et al. (2003) – slope instability; Martin and Lew (1999) – liquefaction; U.S. Army Corps of Engineers (1998) – slope instability; liquefaction; surface fault rupture. More detailed evaluation procedures such as those described below should be used if a hazard cannot be screened out.

Slope instability hazard. 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 ground acceleration, a_{max} or $S_{DS}/2.5$. 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 resulting in estimates of slope displacement are now accepted practice. The most common analysis, termed a Newmark analysis (Newmark, 1965), 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 C7.4-1 for forces and equations used in analysis and Figure C7.4-2 for a schematic illustration for a calculation of the displacement of a soil block toward a bluff.

Acceptable methods for the determination of displacements on many projects involve the use of charts that show displacements for different acceleration ratios, where the acceleration ratio is defined as the ratio of yield acceleration to peak ground acceleration. Various charts have been developed, including those by Franklin and Chang (1977), Makdisi and Seed (1978), Wong and Whitman (1982), Hynes and Franklin (1984), Martin and Qiu (1994); and Bray and Rathje (1998). The selection between the different charts should be made on the basis of the type of slope and the degree of conservatism necessary for the project. A number of the chart methods were developed for the estimation of displacements for dams, and therefore, may be more suitable for embankment designs. Recommendations on the use of such procedures for typical building construction are presented by Blake et al. (2002).



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 and W = weight of soil block

$$F_{rs} = S_u L$$

where S_u = average undrained shear strength of soil and L = length of soil block

Yield seismic coefficient:

$$K_y - \frac{F_{rs} - F_{da}}{W}$$

Figure C7.4-1 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)

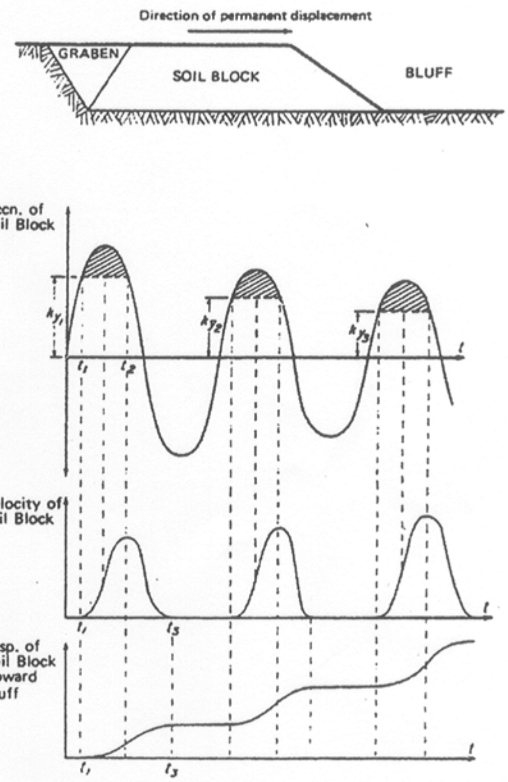


Figure C7.4-2 Schematic illustration for calculating displacement of soil block toward the bluff (National Research Council, 1985; from Idriss, 1985, adapted from Goodman and Seed, 1966)

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, reaction piles or shafts, ground improvement using densification or soil mixing methods, or chemical treatment.

Liquefaction hazard. Liquefaction of saturated granular soils has been a major source of building damage during past earthquakes. Loss of bearing strength, differential settlement, and horizontal displacement due to lateral spreads have been the direct causes of damage. Examples of this damage can be found in reports from many of the more recent earthquakes in the United States, including the 1964 Alaska, the 1971 San Fernando, the 1989 Loma Prieta, the 1994 Northridge, the 2001 Nisqually, and the 2003 Denali earthquakes. Similar damage was reported after the 1964 Niigata, the 1994 Hyogoken-Nanbu (Kobe), the 1999 Taiwan, and the 1999 Turkey earthquakes. As earthquakes occur in the future, additional cases of liquefaction-related damage must be expected. 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 or strength loss, 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 to define a cyclic resistance ratio (CRR) and a cyclic stress ratio induced by the earthquake (CSR).

The following possible methods for calculating the factor of safety against liquefaction have been proposed and used to various extents:

1. Empirical Methods—The most widely used method in practice involves empirical procedures. These procedures rely on correlations between observed cases of liquefaction and measurements made in the field with conventional exploration methods. Seed and Idriss (1971) first published the widely used “simplified procedure” utilizing the Standard Penetration Test (SPT). Since then, the procedure has evolved, primarily through summary papers by Professor H.B. Seed and his colleagues, and field test methods in addition to the SPT have been utilized in similar simplified procedures. These methods include cone penetrometer tests (CPTs), Becker hammer tests (BHTs), and shear wave velocity tests (SVTs). In 1996, a workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) was convened by Professors T.L. Youd and I.M. Idriss with 20 experts to review and update the simplified procedure which had last been updated in 1985. The update of the simplified procedure that resulted from the NCEER workshop (termed herein the “Liquefaction Workshop”) is summarized in NCEER (1997) and in Youd et al. (2001). Martin and Lew (1999) focused on the implementation of this procedure in engineering practice, especially for southern California. The procedure described in NCEER (1997), Youd et al. (2001), and Martin and Lew (1999) using the Standard Penetration Test (SPT) is later summarized in this section.
2. Analytical Methods—Analytical methods are used less frequently to evaluate liquefaction potential – though they may be required for special projects or where soil conditions are not amenable to the empirical method. Analytical methods will also likely gain prominence with time as numerical methods and soil models improve and are increasingly validated. Originally (circa 1970s) the analytical method involved determination of the induced shearing stresses with a program such as SHAKE and comparing these stresses to results of cyclic triaxial or cyclic simple shear tests. Now the analytical method usually refers to a computer code that incorporates a soil model that calculates the buildup in pore water pressure. These more rigorous numerical methods include one-dimensional, nonlinear effective stress codes such as DESRA and SUMDES and two dimensional, nonlinear effective stress codes such as FLAC, TARA, and DYNAFLOW. This new generation of analytical methods has soil models that are fit to laboratory data or liquefaction curves derived from SPT information. The methods are limited by the ability to represent the soil model from either the laboratory or field measurements and by the complexity of the wave propagation mechanisms, including the ability to select appropriate earthquake records to use in the analyses.
3. Physical Modeling—These methods typically involved the use of centrifuges or relatively small-scale shaking tables to simulate seismic loading under well defined boundary conditions. More recently these methods have been expanded to include large laminar boxes mounted on very large

shake tables and full-scale field blast loading tests. Physical modeling of liquefaction is one of the main focus areas of the 2004-2014 Network for Earthquake Engineering Simulation (NEES) supported by the National Science Foundation. Soil used in the small-scale and laminar box models is reconstituted to represent different density and geometrical conditions. Because of difficulties in precisely modeling in-situ conditions at liquefiable sites, small-scale and laminar box 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. Recently, blast loading tests have been conducted to capture the in situ characteristics of the soil for research purposes (e.g., Treasure Island, California and in Japan). However, the cost and safety issues of this approach limits its use to only special design or research projects.

The empirical approach for evaluating liquefaction hazards based on the Liquefaction Workshop and described in NCEER (1997) and Youd it al. (2001) is summarized in the following paragraphs.

The first step in the liquefaction hazard evaluation using the empirical approach is usually to define the normalized cyclic shear stress ratio (CSR) from the peak horizontal ground acceleration expected at the site. This evaluation is made using the following simple equation:

$$CSR = 0.65\left(a_{\max}/g\right)\left(\sigma_o/\sigma'_o\right)r_d \tag{C7.4-1}$$

where (a_{max}/g) = peak horizontal acceleration at ground surface expressed as a decimal fraction of gravity, σ_0 = the vertical total stress in the soil at the depth in question, σ_0' = the vertical effective stress at the same depth, and r_d = deformation-related stress reduction factor.

The peak ground 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. C7.4-1 is the estimated rock acceleration corrected for soil site response but with neglect of excess pore-water pressures that might develop. The acceleration can be determined using the general procedure described in Sec. 3.3 and taking a_{max} equal to $S_{DS}/2.5$. Alternatively, a_{max} can be estimated from: (1) values obtained from the USGS national ground motion maps [see internet website http://geohazards.cr.usgs.gov/eq/] for a selected probability of exceedance, with correction for site effects using the Fa site factor in Sec. 3.3; or (2) from a site-specific ground motion analysis conforming to the requirements of Sec. 3.4.

The stress reduction factor, r_d, used in Eq. C7.4-1 was originally determined using a plot developed by Seed and Idriss (1971) showing the reduction factor versus depth. The consensus from the Liquefaction Workshop was to represent r_d by the following equations:

$$\begin{aligned} r_d &= 1.0 - 0.00765z \text{ for } z \le 9.15 \text{ m} \\ r_d &= 1.174 - 0.267z \text{ for } 9.15\ m < z \le 23 \text{ m} \end{aligned} \tag{C7.4-2}$$

It should be noted that because nearly all the field data used to develop the simplified procedure are for depths less than 12 m, there is greater uncertainty in the evaluations at greater depths. The second step in the liquefaction hazard evaluation using the empirical approach usually involves determination of the normalized cyclic resistance ratio (CRR). The most commonly used empirical relationship for determining CRR was originally compiled by Seed et al. (1985). This relationship compares CRR with corrected Standard Penetration Test (SPT) resistance, $(N_1)_{60}$, from sites where liquefaction did or did not develop during past earthquakes. Figure C7.4-3 shows this relationship for Magnitude 7.5 earthquakes, with an adjustment at low values of CRR recommended by the Liquefaction Workshop. Similar relationships have been developed for determining CRR from CPT soundings, from BHT blowcounts, and from shear wave velocity data, as discussed by Youd et al. (2001) and as presented in detail in NCEER (1997). Only the SPT method is presented herein because of its more common use.

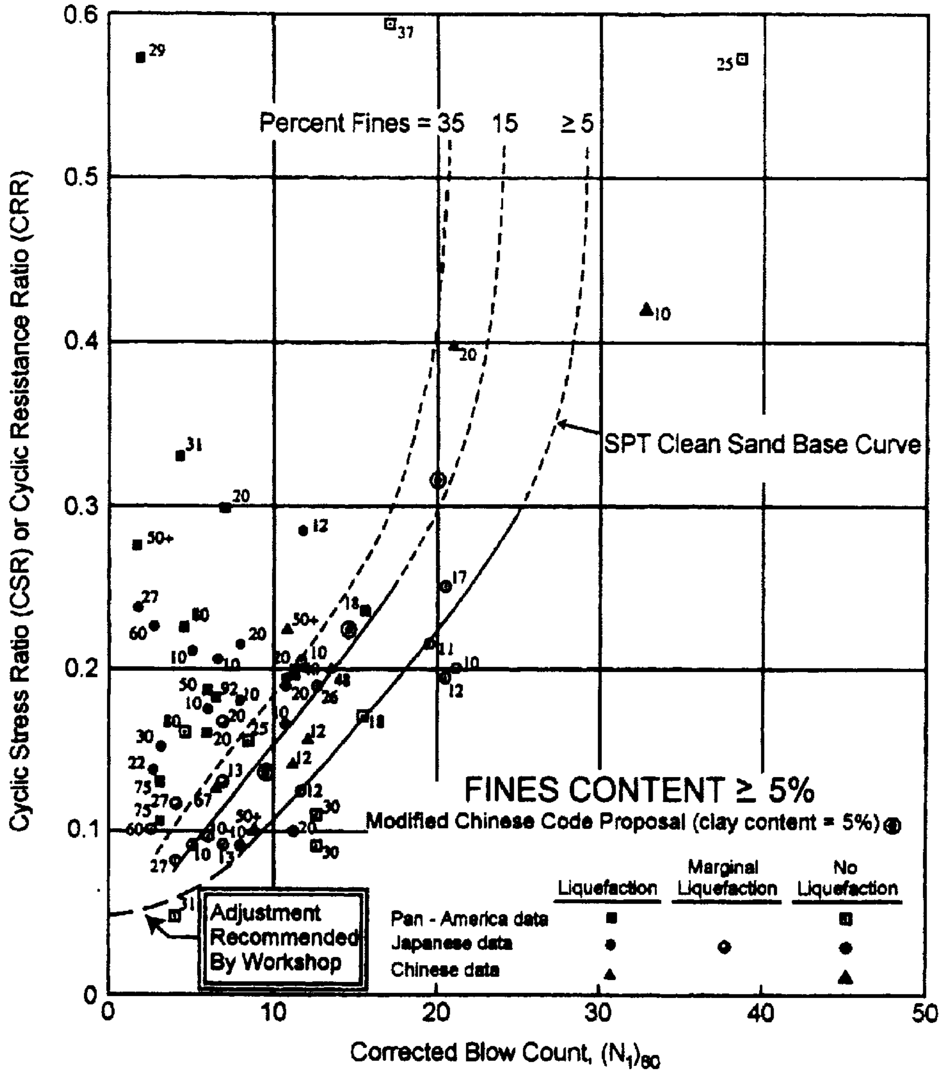


Figure C7.4-3. SPT clean sand base curve for magnitude 7.5 earthquakes with data from liquefaction case histories. (Modified from Seed et al., 1985). (NCEER, 1997; Youd et al., 2001).

In Figure C7.4-3, CRRs calculated for various sites are plotted against $(N_1)_{60}$, where $(N_1)_{60}$ is the SPT blowcount normalized for an overburden stress of 100 kPa and for an energy ratio of 60 percent. Solid symbols represent sites where liquefaction occurred and open symbols 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. As shown, curves are given for soils with fines contents (FC) ranging from less than 5 to 35 percent.

While Figure C7.4-3 provides information about the variation in CRR with fines content, the preferred approach from the Liquefaction Workshop for adjusting for fines is to correct $(N_1)_{60}$ to an equivalent clean sand value, $(N_1)_{60cs}$ using the following equations:

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \quad \text{(C7.4-3)}$$

where α and β = coefficients determined from the following relationships:

$\alpha = 0$ for FC ≤ 5%

$\alpha = \exp[1.76 - (190/FC^2)]$ for 5% < FC < 35%

$\alpha = 5.0$ for FC ≥ 35%

$\beta = 1.0$ for FC ≤ 5%

$\beta = [0.99 + (FC^{1.5}/1,000)$ for 5% < FC < 35%

$\beta = 1.2$ for FC ≥ 35%

Several other corrections are made to $(N_1)_{60}$, as represented in the following equation:

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \quad \text{(C7.4-4)}$$

where N_m = measured standard penetration resistance; C_N = factor to normalize N_m to a common reference effective overburden stress; C_E = correction for hammer energy ratio (ER); C_B = correction factor for borehole diameter; C_R = correction factor for rod length; and C_S = correction for samples with or without liners. Values given in Youd, et al., 2001) are shown in Table C.4-1. An alternative equation for C_n from that shown in the table (Youd, et al., 2001):

$$C_N = 2.2 / [1.2 + \sigma'_{vo}/Pa] \quad \text{(C7.4-5)}$$

where the maximum value of C_N is equal to 1.7. The effective vertical stress, σ'_{vo}, is the stress at the time of the SPT measurement. Youd et al. (2001) caution that other means should be used to evaluate C_N if σ'_{vo} is greater than 300 kPa.

TABLE C7.4-1

Corrections to SPT modified from Skempton 1986) as listed by Robertson and Wride (1998) (from Youd et al., 2001)

Factor	Equipment variable	Term	Correction
Overburden pressure	---	C_N	$(P_a/\sigma'_{vo})^{0.5}$
Overburden pressure	---	C_N	$C_N < 1.7$
Energy ratio	Donut hammer	C_E	0.5 – 1.0
Energy ratio	Safety hammer	C_E	0.7 – 1.2
Energy ratio	Automatic-trip Donut-type hammer	C_E	0.8 – 1.3
Borehole diameter	65 – 115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	< 3	C_R	0.75
Rod length	3 – 4 m	C_R	0.8
Rod length	4 -6 m	C_R	0.85
Rod length	6 – 10 m	C_R	0.95
Rod length	10 – 30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler without liners	C_S	1.1 – 1.3

It is very important that the engineer consider these correction factors when conducting the liquefaction analyses. Failure to consider these corrections can result in inaccurate liquefaction estimates – leading to either excessive cost to mitigate the liquefaction concern or excessive risk of poor performance during a design event – potentially resulting in unacceptable damage.

Special mention also needs to be made of the energy calibration term, C_E. This correction has a very significant effect on the $(N_1)_{60}$ used to compute CRR. The value of this correction factor can vary greatly depending on the SPT hammer system used in the field and on site conditions. For important sites where C_E could result in changes from liquefied to non liquefied, energy ratio measurements should be made. These measurements are relatively inexpensive and represent a small increase in overall field exploration costs. Many drilling contractors in areas that are seismically active provide calibrated equipment as part of their routine service.

Before computing the factor of safety from liquefaction, the CRR result obtained from Figure C7.4-3 (using the corrected SPT blow count identified in the equation for $(N_1)_{60}$) must be corrected for earthquake magnitude M if the magnitude differs from 7.5. The magnitude correction factor is shown in Figure C7.4-4. This plot was developed during the Liquefaction Workshop on the basis of input from experts attending the workshop. The range shown in Figure C7.4-4 is used because of uncertainties. The user should select a value consistent with the project risk. For M greater than 7.5 the factors recommended by Idriss (second from highest) should be used.

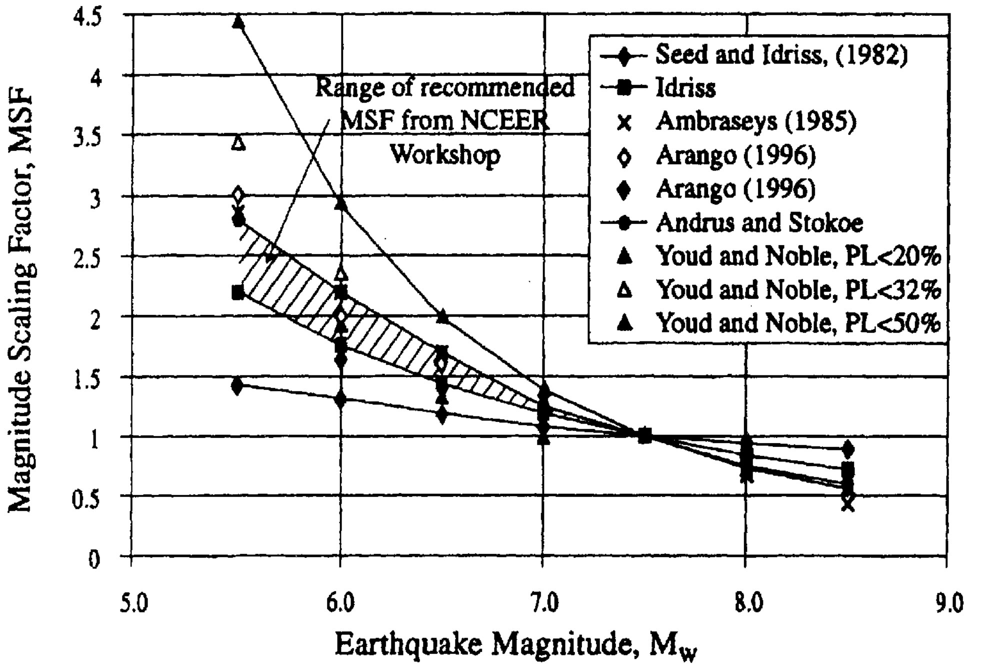


Figure C7.4-4. Magnitude scaling factors derived by various investigators. (NCEER, 1997; Youd et al., 2001)

The magnitude, M, needed to determine a magnitude scaling factor from Figure C7.4-4 should correspond to the Maximum Considered Earthquake (MCE). Where the general procedure for ground motion estimation is used (Sec. 3.3) and the MCE is determined probabilistically, the magnitude used in these evaluations can be obtained from deaggregation information available by latitude and longitude from the USGS website (http://geohazards.cr.usgs.gov/eq/). Where the general procedure (Sec. 3.3) is used and the MCE is bounded deterministically near known active fault sources (*Commentary* Appendix A), the magnitude of the MCE should be the characteristic maximum magnitude assigned to the fault in the construction of the MCE ground motion maps. Where the site-specific procedure for ground motion estimation is used (Sec. 3.4), the magnitude of the MCE should be similarly determined from the site-specific analysis. 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. C7.4-6) is determined jointly by a_{max} and M and not by a_{max} alone. Because of the longer duration of strong ground-shaking, large distant earthquakes may in some cases 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 final step in the liquefaction hazard evaluation using the empirical approach involves the computation of the factor of safety (F_L) against liquefaction using the equation:

$$F_L = \text{CRR/CSR} \tag{C7.4-6}$$

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. Although the curves shown in Figure C7.4-3 envelop the plotted data, it is possible that liquefaction may have occurred beyond the enveloped data and was not detected at ground surface. For this reason a factor of safety of 1.2 to 1.5 is usually appropriate for building sites – with the actual factor selected on the basis of the importance of the structure and the potential for ground displacement at the site.

Additional guidance on the selection of the appropriate factor of safety is provided by Martin and Lew 1999. They suggest that the following factors be considered when selecting the factor of safety:

1. The type of structure and its vulnerability to damage.
2. Levels of risk accepted by the owner or governmental regulations with questions related to design for life safety, limited structural damage, or essentially no damage.
3. Damage potential associated with the particular liquefaction hazards. Flow failures or major lateral spreads pose more damage potential than differential settlement. Hence factors of safety could be adjusted accordingly.
4. Damage potential associated with design earthquake magnitude. A magnitude 7.5 event is potentially more damaging than a 6.5 event.
5. Damage potential associated with SPT values; low blow counts have a greater cyclic strain potential than higher blowcounts.
6. Uncertainty in SPT- or CPT- derived liquefaction strengths used for evaluations. Note that a change in silt content from 5 to 15 percent could change a factor of safety from, say, 1.0 to 1.25.
7. For high levels of design ground motion, factors of safety may be indeterminate. For example, if $(N_1)_{60}$ = 20, M = 7.5, and fines content = 35 percent, liquefaction strengths cannot be accurately defined due to the vertical asymptote on the empirical strength curve.

Martin and Lew (1999) indicate that the final choice of an appropriate factor of safety must reflect the particular conditions associated with the specific site and the vulnerability of site-related structures. Table C7.4-2 summarizes factors of safety suggested by Martin and Lew.

Table C7.4-2. Factors of safety for liquefaction hazard assessment (from Martin and Lew, 1999).

Consequences of Liquefaction	$(N_1)_{60cs}$	Factor of Safety
Settlement	≤ 15	1.1
	≥ 30	1.0
Surface Manifestations	≤ 15	1.2
	≥ 30	1.0
Lateral Spread	≤ 15	1.3
	≥ 30	1.0

As a final comment on the assessment of liquefaction hazards, it is important to note that soils composed of sands, silts, and gravels are most susceptible to liquefaction while clayey soils generally are not susceptible to this phenomenon. The curves in Figure C7.4-3 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 C7.4-3 for 35 percent fines should be used provided the following criteria are met (Seed and Idriss, 1982; Seed et al., 1983): 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. If these criteria are not met, the soils may be considered nonliquefiable.

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. Surface manifestations, loss of bearing strength, ground settlement, flow failure and lateral spread are ground failure mechanisms that have caused structural damage during past earthquakes. These types of ground failure are described in Martin and Lew (1999), U.S. Army Corps of Engineers (1998) and National Research Council (1985) and are discussed below. The type of failure and amount of ground displacement are a function of several parameters including the looseness of the liquefied soil layer, the thickness and extent of the liquefied layer, the thickness and permeability of unliquefied material overlying the liquefied layer, the ground slope, and the nearness of a free face.

*Surface Manifestations***.** Surface manifestations refer to sand boils and ground fissures on level ground sites. For structures supported on shallow foundations, the effects of surface manifestations on the structure could be tilting or cracking. Criteria are given by Ishihara (1985) for evaluating the influence of thickness of layers on surface manifestation of liquefaction effects for level sites. These criteria may be used for noncritical or nonessential structures on level sites not subject to lateral spreads (see later in this section). Additional analysis should be performed for critical or essential structures.

Loss of bearing strength. Loss of bearing strength can occur if the foundation is located within or above the liqefiable layer. The consequence of bearing failure could be settlement or tilting of the structure. Usually, 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. Simple guidance for how deep or how thin the layers must be has not yet been developed. Martin and Lew (1999) provide some preliminary guidance based on the Ishihara (1985) method. Final evaluation of the potential for loss of bearing strength should be made by a geotechnical engineer experienced in liquefaction hazard assessment

Ground settlement. For saturated or dry granular soils in a loose condition, the amount of ground settlement could approach 3 to 4 percent of the thickness of the loose soil layer in some cases. This amount of settlement could cause tilting or cracking of a building, and therefore, it is usually important to evaluate the potential for ground settlement during earthquakes.

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. The Tokimatsu and Seed procedure can be applied whether liquefaction does or does not occur. For dry cohesionless soils, the settlement estimate from Tokimatsu and Seed should be multiplied by a factor of 2 to account for multi-directional shaking effects as discussed by Martin and Lew (1999).

*Flow failures***.** Flow failures or flow slides are the most catastrophic form of ground failure that may be triggered when liquefaction occurs. They may displace large masses of soils tens of meters. Flow slides occur when the average static shear stresses on potential failure surfaces are less than the average shear strengths of liquefied soil on these surfaces. Standard limit equilibrium static slope stability analyses may be used to assess flow failure potential with the residual strength of liquefied soil used as the strength parameter in the analyses.

The determination of residual strengths is very inexact, and consensus as to the most appropriate approach has not been reached to date. Two relationships for residual strength of liquefied soil that are often used in practice are those of Seed and Harder (1990) and Stark and Mesri (1992). A more complete discussion and references on this topic may be found in Martin and Lew (1999).

Lateral spreads. Lateral spreads are ground-failure phenomena that can occur on gently sloping ground underlain by liquefied soil. They may result in lateral movements in the range of a few centimeters to several meters. Earthquake ground-shaking affects the stability of gently sloping ground containing liquefiable materials by seismic inertia forces combined with static gravity forces within the slope and by shaking-induced strength reductions in the liquefiable materials. Temporary instability due to seismic inertia forces are manifested by lateral "downslope" movement. For the duration of ground shaking associated with moderate-to large-magnitude earthquakes, there could be many such occurrences of temporary instability during earthquake shaking, producing an accumulation of "downslope" movement.

Various analytical and empirical techniques have been developed to date to estimate lateral spread ground displacement; however, no single technique has been widely accepted or verified for engineering design. Three approaches are used depending on the requirements of the project: empirical procedures, simplified analytical methods, and more rigorous computer modeling. Empirical procedures use correlations between past ground displacement and site conditions under which those displacements occurred. Youd et al. (2002) present an empirical method that provides an estimate of lateral spread displacements as a function of earthquake magnitude, distance, topographic conditions, and soil deposit characteristics. As shown in Figure C7.4-5, the displacements estimated by the Youd et al. (2002) method are generally within a factor of two of the observed displacements. Bardet et al. (2002) present an empirical method having a formulation similar to that of Youd et al. (2002) but using fewer parameters to describe the soil deposit. The Bardet et al. (2002) model was developed to assess lateral spread displacements at a regional scale rather than for site-specific applications.

Simplified analytical techniques generally apply some form of 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. Additional discussion of the simplified Newmark method is provided in Sec. 7.4.1. More rigorous computer modeling typically involves use of nonlinear finite element or finite difference methods to predict deformations, such as with the computer code FLAC. Both the simplified Newmark method and the rigorous computer codes require considerable experience to obtain meaningful results. For example, the soil model within the nonlinear computer codes is often calibrated for only specific conditions. If the site is not characterized by these conditions, errors in estimating the displacement by a factor of two or more can easily occur.

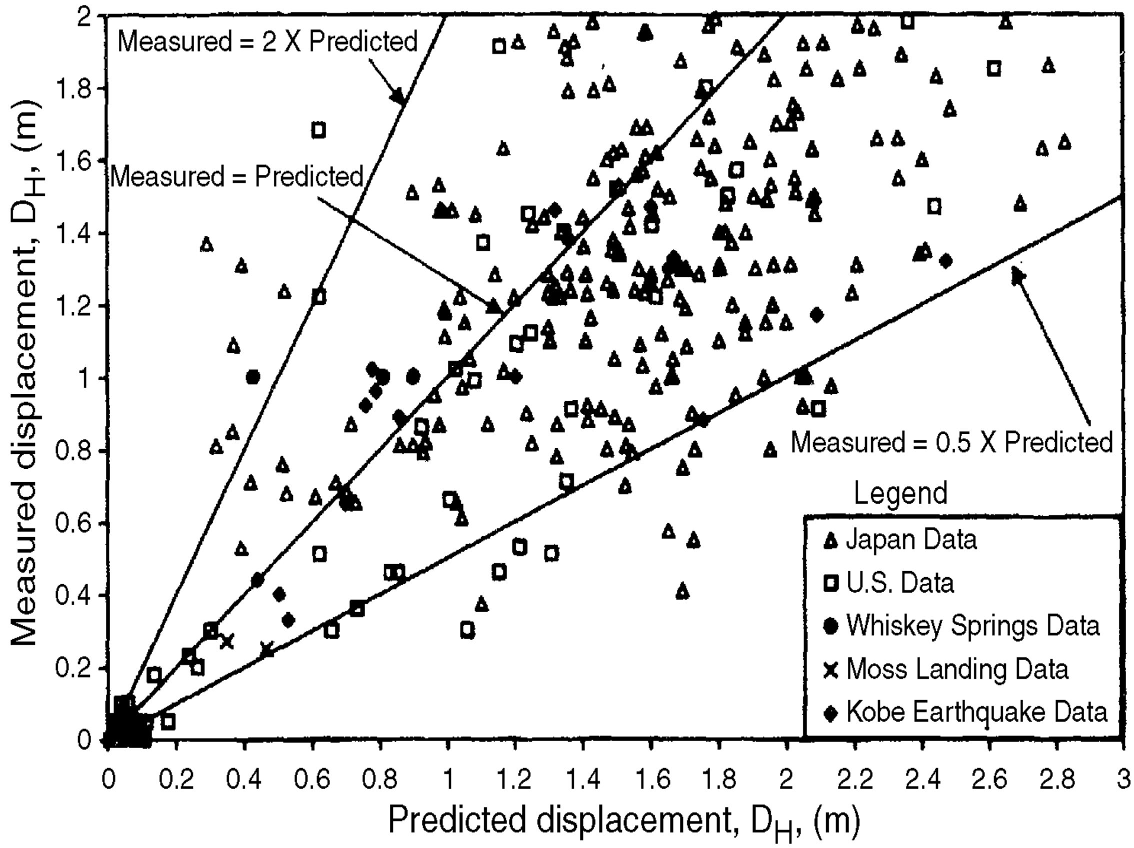


Figure C7.4-5. Measured versus predicted displacements for displacements up to 2 meters. (Youd et al., 2002).

Liquefaction-induced deformations are not directly proportional to ground motions and may be more than 50 percent higher for maximum considered earthquake ground motions than for design earthquake ground motions. The liquefaction potential and resulting deformations for ground motions consistent with the maximum considered earthquake should also be evaluated and, while not required in the *Provisions*, should be used by the registered design professional in checking for building damage that may result in collapse. In addition, Seismic Use Group III structures should be checked for their required post-earthquake condition.

Mitigation of liquefaction hazard. With respect to the hazard of liquefaction, 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. 7.4.3. 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 reduced 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 ft, although re-leveling 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 foot.

Evaluations of structural performance following two recent Japanese earthquakes, 1993 Hokkaido Nansei-Oki and 1995 (Kobe) Hyogo-Ken Nanbu, indicate that small structures on shallow foundations performed well in liquefaction areas. Sand boil eruptions and open ground fissures in these areas indicate minor effects of liquefaction, including ground oscillation and up to several tenths of a meter of lateral spread displacement. Many small structures (mostly houses, shops, schools, etc.) were structurally undamaged although a few tilted slightly. Foundations for these structures consist of reinforced concrete perimeter wall footings with reinforced concrete interior wall footings tied into the perimeter walls at intersections. These foundations acted as diaphragms causing the soil to yield beneath the foundation which prevented fracture of foundations and propagation of differential displacements into the superstructure.

Similarly, well reinforced foundations that would not fracture could be used in U.S. practice as a mitigative measure to reduce structural damage in areas subject to liquefaction but with limited potential for lateral (< 0.3 m) or vertical (< 0.05 m) ground displacements. Such strengthening also would serve as an effective mitigation measure against damage from other sources of limited ground displacement including fault zones, landslides, and cut fill boundaries. Where slab-on-grade or basement slabs are used as foundation elements, these slabs should be reinforced and tied to the foundation walls to give the structure adequate strength to resist ground displacement. Although strengthening of foundations, as noted above, would largely mitigate damage to the structure, utility connections may be adversely affected unless special flexibility is built into these nonstructural components.

Another possible consequence of liquefaction to structures is increased lateral pressures against basement walls as discussed in Sec. 7.5.1. A common procedure used in design for such increased pressures is to assume that the liquefied material acts as a dense fluid having a unit weight of the liquefied soil. The wall then is designed assuming that hydrostatic pressure for the dense fluid acts along the total subsurface height of the wall. The procedure applies equivalent horizontal earth pressures that are greater than typical at-rest earth pressures but less than passive earth pressures. As a final consideration, to prevent buoyant rise as a consequence of liquefaction, the total weight of the structure should be greater than the volume of the basement or other cavity times the unit weight of liquefied soil. (Note that structures with insufficient weight to counterbalance buoyant effects could differentially rise during an earthquake.)

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).

Surface fault rupture hazard. 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. Sources of detailed information for evaluating the hazard of surface fault rupture include Slemmons and dePolo (1986), the Utah Section of the Association of Engineering Geologists (1987), Swan et al. (1991), Hart and Bryant (1997), and California Geological Survey (2002). Other beneficial references are given in the bibliographies of these publications.

Assessment of surface faulting hazard. The evaluation of surface fault rupture 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 active 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 related to foundation design, a sufficient definition of an active fault is given in CDMG Special Publication 42 (Hart and Bryant, 1997): "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, the amount and character of past displacements, and the expected amount and potential of future displacement. 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 often 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/geomorphic features, vegetation and soil contrasts, and other lineaments of possible fault origin. The study of predevelopment aerial photographs is often essential to the detection of fault features.

3. A field reconnaissance study generally is required and should include observation and mapping of bedrock and soil units and structures, geomorphic surfaces, fault-related geomorphic features, springs, and deformation of man-made structures due to fault creep. Field 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 location and activity of fault traces. These investigations may include trenches, test pits, and/or boreholes to permit detailed and direct observation of geologic units and faults.

5. The geometry of faults 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 typically 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 (C^{14}, K-Ar), stratigraphic correlation (fossils, mineralology) soil profile development, paleomagnetism (magnetostratigraphy), or other dating techniques (thermoluminescense, cosmogenic isotopes) to date the age of faulted or unfaulted units or surfaces. Probabilistic studies may be considered to evaluate the probability of fault displacement (Youngs et al., 2003).

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. The distribution of primary and secondary faulting (fault zone width) and fault-related surface deformation should be shown.

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

3. From this documentation, estimates of location, magnitude, and likelihood or relative potential for future fault displacement 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 Wells and Coppersmith (1994). 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).

4. The degree of confidence and limitations of the data should be addressed.

There are no codified procedures for estimating the amount or probability of future fault displacements. Estimates may be made, however, by qualified earth scientists using techniques described above. 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 ft 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 Code of Regulations*, Title 14, Division 2, Sec. 3603(a)).
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-ft 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; Kelson et al., 2001). 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.

7.4.2 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.

7.4.3 Foundation ties. One of the prerequisites of adequate performance of a building during an earthquake is the provision of a foundation that acts as a unit and does not permit one column or wall to move appreciably with respect to another. A common method used to attain this is to provide ties between footings and pile caps. This is especially necessary where the surface soils are soft enough to require the use of piles or caissons. Therefore, the pile caps or caissons are tied together with nominal ties capable of carrying, in tension or compression, a force equal to S_{DS}/10 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 (such as 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 (such as 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 (that is, the point where the flow of the soil around the piles may be negligible).

7.4.4 Special pile requirements. Special requirements for piles, piers, or caissons in Seismic Design Category C are given in this section. Provisions for pile anchorage to the pile cap or grade beam and transverse reinforcement detailing requirements for concrete piles are provided. The anchorage requirements are intended to assure that the connection to the pile cap does not fail in a brittle manner under moderate ground motions. Moderate ground motions could result in pile tension forces or bending moments which could compromise shallow anchorage embedment. Shallow anchorages in pile caps may consist of short lengths of reinforcing bars or bare structural steel pile sections. Loss of pile anchorage could result in unintended increases in vertical seismic force resisting element drifts from rocking, potential overturning instability of the superstructure, and loss of shearing resistance at the ground surface. Anchorage by shallow embedment of the bare steel pile section is not recommended due to the degradation of the concrete bond from cracking as a result of the cyclic loading from the moderate ground motions. Exception to this is permitted for steel pipe piles filled with concrete when the connection is made with reinforcing bar dowels properly developed into the pile and pile cap. The confinement of the interior concrete by the “hoop” stresses of the circular pile section was judged to be sufficient to prevent concrete pullout from that section. Using this method of connection, the structural steel pipe section should be embedded into the pile cap for a short distance or else the pile should be designed as an uncased concrete pile. End anchorage detailing requirements for transverse reinforcement generally follow that required by ACI 318, Chapter 21 to assure that no loss of confinement of the transverse reinforcement occurs in concrete piles since verification of pile damage after moderate ground motions is difficult or not done.

7.4.4.1 Uncased concrete piles. The uncased concrete piles category has been expanded to include auger-cast piles which are now subject to the same reinforcement requirements as other cast-in-place concrete piles. The longitudinal reinforcement within a pile has prescriptive termination point minimums intended for firm soils and is required to extend at least the full flexural length. The longitudinal reinforcement should extend past the flexural length by its development length requirement. The flexural length has been defined as that length of a pile from bottom of pile cap to a point where 0.4 times the concrete section cracking moment exceeds the calculated flexural demand. The 0.4 factor is analogous to a material resistance factor in strength design. This definition implies the plain concrete section will be resisting some minimal amount of moment demand beyond the “flexural” length. Where the pile is subject to significant uplift forces, it is recommended that the longitudinal reinforcement extend the full length of the pile.

Increased transverse reinforcement requirements are given for the potential plastic hinge zone immediately below the pile cap. The potential plastic hinge zone was taken to be three pile diameters below the pile cap to allow for varied soil site classes. The transverse reinforcement detailing for this zone is similar to that required for concrete intermediate moment frames at hinge regions and is expected to provide a displacement ductility of approximately 4. Beyond the potential plastic hinge region, the curvature ductility demand is not considered to exceed that provided by the nominal moment capacity of the section for non-earthquake loads.

7.4.4.4 Precast (non-prestressed) concrete piles. For precast concrete piles, the longitudinal reinforcement is specified to extend the full length of the pile so there is no need to determine the flexural length. Transverse reinforcement spacing within the potential plastic hinge zone is required for the length of three pile diameters at the bottom of pile cap. Particular attention should be taken where piles cannot be driven to or are overdriven beyond the anticipated end bearing point elevation. The transverse reinforcement size and spacing in this region is the same as the uncased concrete pile. Transverse reinforcement spacing outside the potential plastic hinge zone is specified to be no greater than 8 inches to conform with current building code minimums for this pile type.

7.4.4.5 Precast-prestressed piles. The transverse reinforcement requirements are primarily taken from the PCI Committee Report (1993) on precast prestressed concrete piling for geographic regions subject to low to moderate ground motions. The amount of transverse reinforcement was relaxed for the pile region greater than 20 feet (6m) below the pile cap to one-half of that required above. It was judged that the reduced transverse reinforcement would be sufficient to resist the reduced curvature demands at that point. Particular attention should be taken where piles cannot be driven to or are overdriven beyond the anticipated end bearing point elevation so that the length of the confining transverse reinforcement is maintained.

Equation (7.4-1), originally from ACI 318, has always been intended to be a lower bound spiral transverse reinforcement ratio for larger diameter columns. It is independent of the member section properties and can therefore be applied to large or small diameter piles. For cast-in-place piles and prestressed concrete piles, the resulting spiral reinforcement ratios from this formula are considered to be sufficient to provide moderate ductility capacities.

High strength hard drawn wire with higher yield strengths is permitted to be used for transverse circular spiral reinforcement of precast prestressed concrete piles. Pile test specimens using this type of transverse reinforcement include the research done by Park and Hoat Joen (1990). High strength hard drawn wire has yield strengths between 150 and 200 ksi. f_{yh} is conservatively limited to 85 ksi for this steel because hard drawn wire has limited ductility.

7.5 SEISMIC DESIGN CATEGORIES D, E, AND F

For Seismic Design Category D, E, or F construction, all the preceding provisions for Seismic Design Category C applies for the foundations, but the earthquake detailing is generally more severe and demanding.

7.5.1 Investigation. In addition to the potential site hazards discussed in *Provisions* Sec. 7.4.1, consideration of lateral pressures on earth retaining structures shall be included in investigations for Seismic Design Categories D, E, and F.

Earth retaining structures. Increased lateral pressures on retaining structures during earthquakes have long been recognized; however, design procedures have not been prescribed in U.S. model building codes. Waterfront structures often have performed poorly in major earthquake due to excess pore water pressure and liquefaction conditions developing in relatively loose, saturated granular soils. Damage reports for structures away from waterfronts are generally limited with only a few cases of stability failures or large permanent movements (Whitman, 1991). Due to the apparent conservatism or overstrength in static design of most walls, the complexity of nonlinear dynamic soil-structure interaction, and the poor understanding of the behavior of retaining structures with cohesive or dense granular soils, Whitman (1991) recommends that “engineers must rely primarily on a sound understanding of fundamental principles and of general patterns of behavior.”

Seismic design analysis of retaining walls is discussed below for two categories of walls: “yielding” walls that can move sufficiently to develop minimum active earth pressures and “nonyielding” walls that do not satisfy this movement condition. The amount of movement to develop minimum active pressure is very small. A displacement at the top of the wall of 0.002 times the wall height is typically sufficient to develop the minimum active pressure state. Generally, free-standing gravity or cantilever walls are considered to be yielding walls (except massive gravity walls founded on rock), whereas building basement walls restrained at the top and bottom are considered to be nonyielding.

Yielding walls. At the 1970 Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, Seed and Whitman (1970) made a significant contribution by reintroducing and reformulating the Monobe-Okabe (M-O) seismic coefficient analysis (Monobe and Matsuo, 1929; Okabe, 1926), the earliest method for assessing the dynamic lateral pressures on a retaining wall. The M-O method is based on the key assumption that the wall displaces or rotates outward sufficiently to produce the minimum active earth pressure state. The M-O formulation is expressed as:

$$P_{AE} = (1/2)\gamma H^2 (1 - k_v) K_{AE} \tag{C7.5-1}$$

where: P_{AE} is the total (static + dynamic) lateral thrust, γ is unit weight of backfill soil, H is height of backfill behind the wall, k_V is vertical ground acceleration divided by gravitational acceleration, and K_{AE} is the static plus dynamic lateral earth pressure coefficient which is dependent on (in its most general form) angle of friction of backfill, angle of wall friction, slope of backfill surface, and slope of back face of wall, as well as horizontal and vertical ground acceleration. The formulation for K_{AE} is given in textbooks on soil dynamics (Prakash, 1981; Das, 1983; Kramer, 1996) and discussed in detail by Ebeling and Morrison (1992).

Seed and Whitman (1970), as a convenience in design analysis, proposed to evaluate the total lateral thrust, P_{AE}, in terms of its static component (P_A) and dynamic incremental component (ΔP_{AE}):

$$P_{AE} = P_A + \Delta P_{AE} \tag{C7.5-2a}$$

or

$$K_{AE} = K_A + \Delta K_{AE} \tag{C7.5-2b}$$

or

$$\Delta P_{AE} = (1/2)\gamma H^2 \Delta K_{AE} \tag{C7.5-2c}$$

Seed and Whitman (1970), based on a parametric sensitivity analysis, further proposed that for practical purposes:

$$\Delta K_{AE} = (3/4) K_h \tag{C7.5-3a}$$

$$\Delta P_{AE} = (1/2)\gamma H^2 (3/4) k_h = (3/8) k_h \gamma H^2 \tag{C7.5-3b}$$

where k_h is horizontal ground acceleration divided by gravitational acceleration. It is recommended that k_h be taken equal to the site peak ground acceleration that is consistent with design earthquake ground motions as determined in *Provisions* Sec. 7.5.2 (that is, $k_h = S_{DS}/2.5$). Eq. C7.5-3a and C7.5-3b generally are referred to as the simplified M-O formulation.

Since its introduction, there has been a consensus in geotechnical engineering practice that the simplified M-O formulation reasonably represents the dynamic (seismic) lateral earth pressure increment for yielding retaining walls. For the distribution of the dynamic thrust, ΔP_{AE}, Seed and Whitman (1970) recommended that the resultant dynamic thrust act at $0.6H$ above the base of the wall (that is, inverted trapezoidal pressure distribution).

Using the simplified M-O formulation, a yielding wall may be designed using either a limit-equilibrium force approach (conventional retaining wall design) or an approach that permits movement of the wall up to tolerable amounts. Richards and Elms (1979) introduced a method for seismic design analysis of yielding walls considering translational sliding as a failure mode and based on tolerable permanent displacements for the wall. There are a number of empirical formulations for estimating permanent displacements under a translation mode of failure; these have been reviewed by Whitman and Liao (1985). Nadim (1980) and Nadim and Whitman (1984) incorporated the failure mode of wall tilting as well as sliding by employing coupled equations of motion, which were further formulated by

Siddharthan et al. (1992) as a design method to predict the seismic performance of retaining walls taking into account both sliding and tilting. Alternatively, Prakash and others (1995) described design procedures and presented design charts for estimating both sliding and rocking displacements of rigid retaining walls. These design charts are the results of analyses for which the backfill and foundation soils were modeled as nonlinear viscoelastic materials. A simplified method that considers rocking of a wall on a rigid foundation about the toe was described by Steedman and Zeng (1996) and allows the determination of the threshold acceleration beyond which the wall will rotate. A simplified procedure for evaluating the critical threshold accelerations for sliding and tilting was described by Richards and others (1996).

Application of methods for evaluating tilting of yielding walls has been limited to a few case studies and back-calculation of laboratory test results. Evaluation of wall tilting requires considerable engineering judgment. Because the tilting mode of failure can lead to instability of a yielding retaining wall, it is suggested that this mode of failure be avoided in the design of new walls by proportioning the walls to prevent rotation in order to displace only in the sliding mode.

Nonyielding walls. Wood (1973) analyzed the response of a rigid nonyielding wall retaining a homogeneous linear elastic soil and connected to a rigid base. For such conditions, Wood established that the dynamic amplification was insignificant for relatively low-frequency ground motions (that is, motions at less than half of the natural frequency of the unconstrained backfill), which would include many or most earthquake problems.

For uniform, constant k_h applied throughout the elastic backfill, Wood (1973) developed the dynamic thrust, ΔP_E, acting on smooth rigid nonyielding walls as:

$$\Delta P_E = F k_h \gamma H^2 \qquad \text{(C7.5-4a)}$$

The value of F is approximately equal to unity (Whitman, 1991) leading to the following approximate formulation for a rigid nonyielding wall on a rigid base:

$$\Delta P_E = k_h \gamma H^2 \qquad \text{(C7.5-4b)}$$

As for yielding walls, the point of application of the dynamic thrust is taken typically at a height of $0.6H$ above the base of the wall.

It should be noted that the model used by Wood (1973) does not incorporate any effect on the pressures of the inertial response of a superstructure connected to the top of the wall. This effect may modify the interaction between the soil and the wall and thus modify the pressures from those calculated assuming a rigid wall on a rigid base. The subject of soil-wall interaction is addressed in the following sections. This section also provides further discussion on the applicability of the Wood and the M-O formulations.

Soil-structure-interaction approach and modeling for wall pressures. Lam and Martin (1986), Soydemir and Celebi (1992), Veletsos and Younan (1994a and 1994b), and Ostadan and White (1998), among others, argue that the earth pressures acting on the walls of embedded structures during earthquakes are primarily governed by soil-structure interaction (SSI) and, thus, should be treated differently from the concept of limiting equilibrium (that is, M-O method). Soil-structure interaction includes both a kinematic component—the interaction of a massless rigid wall with the adjacent soil as modeled by Wood (1973)—and an inertial component—the interaction of the wall, connected to a responding superstructure, with the adjacent soil. Detailed SSI analyses incorporating kinematic and inertial interaction may be considered for the estimation of seismic earth pressures on critical walls. Computer programs that may be utilized for such analyses include FLUSH (Lysmer et. al, 1975) and SASSI (Lysmer et al., 1981).

Ostadan and White (1998) have observed that for embedded structures subjected to ground shaking, the characteristics of the wall pressure amplitudes vs. frequency of the ground motion were those of a single-degree-of-freedom (SDOF) system and proposed a simplified method to estimate the magnitude and distribution of dynamic thrust. Results provided by Ostadan and White (1998) utilizing this simplified method, which were also confirmed by dynamic finite element analyses, indicate that, depending on the dynamic properties of the backfill as well as the frequency characteristics of the input ground motion, a range of dynamic earth pressure solutions would be obtained for which the M-O solution and the Wood (1973) solution represent a "lower" and an "upper" bound, respectively.

Chang and others (1990) have found that dynamic earth pressures recorded on the wall of a model nuclear reactor containment building were consistent with dynamic pressures predicted by the M-O solution. Analysis by Chang and others indicated that the dynamic wall pressures were strongly correlated with the rocking response of the structure. Whitman (1991) has suggested that SSI effects on basement walls of buildings reduce dynamic earth pressures and that M-O pressures may be used in design except where structures are founded on rock or hard soil (that is, no significant rocking). In the latter case, the pressures given by the Wood (1973) formulation would appear to be more applicable. The effect of rocking in reducing the dynamic earth pressures on basement walls also has been suggested by Ostadan and White (1998). This condition may be explained if it is demonstrated that the dynamic displacements induced by kinematic and inertial components are out of phase.

Effect of saturated backfill on wall pressures. The previous discussions are limited to backfills that are not water-saturated. In current (1999) practice, drains typically are incorporated in the design to prevent groundwater from building up within the backfill. This is not practical or feasible, however, for waterfront structures (such as quay walls) where most of the earthquake-induced failures have been reported (Seed and Whitman, 1970; Ebeling and Morrison, 1992; ASCE-TCLEE, 1998).

During ground shaking, the presence of water in the pores of a backfill can influence the seismic loads that act on the wall in three ways (Ebeling and Morrison, 1992; Kramer, 1996): (1) by altering the inertial forces within the backfill, (2) by developing hydrodynamic pressures within the backfill and (3) by generating excess porewater pressure due to cyclic straining. Effects of the presence of water in cohesionless soil backfill on seismic wall pressures can be estimated using formulations presented by Ebeling and Morrison (1992).

A soil liquefaction condition behind a wall may under the design earthquake have a pronounced effect on the wall pressures during and for some time after the earthquake. At present (1999), there is no general consensus established for estimating lateral earth pressures for liquefied backfill conditions. One simplified and probably somewhat conservative approach is to treat the liquefied backfill as a heavy viscous fluid exerting a hydrostatic pressure on the wall. In this case, the viscous fluid has the total unit weight of the liquefied soil. If unsaturated soil is present above the liquefied soil, it is treated as a surcharge that increases the fluid pressure within the underlying liquid soil by an amount equal to the thickness times the total unit weight of the surcharge soil. In addition to these "static" fluid pressures exerted by a liquefied backfill, hydrodynamic pressures can be exerted by the backfill. The magnitude of any such hydrodynamic pressures would depend on the level of shaking following liquefaction. Hydrodynamic effects may be estimated using formulations presented by Ebeling and Morrison (1992).

7.5.3 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. 7.4.3 also applies here.

7.5.4 Special pile and grade beam requirements. For Seismic Design Categories D, E, and F, additional pile reinforcement over that specified for Seismic Design Category C buildings is required. Adequate pile ductility is required and provision must be made for additional reinforcement to ensure, as a minimum, full ductility in the upper portion of the pile.

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 are supported in soils such as loose granular materials and/or soft soils that are susceptible to large deformations and/or strength degradation. Severe pile bending problems may result from various combinations of soil conditions during strong ground shaking, for example:

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. Analysis techniques to evaluate pile bending are discussed by Margason and Holloway (1977) and Mylonakis (2001) and these effects on concrete piles are further discussed by Shepard (1983). For homogeneous, elastic media and assuming the pile follows the soil, the free-field curvature (soil strains without a pile present) can be estimated by dividing the peak ground acceleration by the square of the shear wave velocity of the soil, although considerable judgment is necessary in utilizing this simple relationship in a layered, inelastic profile with pile-soil interaction effects. Norris (1994) discusses methods to assess pile-soil interaction with regard to pile foundation behavior.

The designer needs to consider the variation in soil conditions and driven pile lengths in providing for pile ductility at potential high curvature interfaces. Interaction between the geotechnical and structural engineers is essential.

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.

Design of piles incorporates the same *R* force reduction factor as the superstructure and therefore implies inelasticity in the foundations and piles. Therefore, piles should be designed with similar ductility requirements as the superstructure. Foundations in SDC D, E, and F are expected to experience strong ground motions and large pile curvatures. Inertial pile-soil-structure interaction may produce plastic hinging in the piles near the bottom of the pile cap. Kinematic soil-pile-structure interaction will result in some bending moments and shearing forces throughout the length of the pile and will be higher at interfaces between stiff and soft soil strata. Inertial pile-soil-structure interaction will be particularly severe in soft soils and liquefiable soils located near the pile cap. This could result in plastic hinging of the pile in reverse curvature near the pile cap and for this reason the potential plastic hinge region is extended to seven pile diameters from the pile cap in the *Provisions*.

Precast prestressed concrete piles are exempted from the concrete special moment frame detailing requirements adapted for concrete piles since these provisions were never intended for slender precast prestressed concrete elements and will result in unbuildable piles. Piles with substantially less confinement reinforcement than required by ACI 318 equation 10-6 have been proven through cyclic testing to have adequate performance (Park and Hoat Joen, 1990). Transverse steel requirements for the precast prestressed concrete piles are given in Section 7.5.4.4.

Where grade beams have the strength to resist the load combination with overstrength, which simulates expected foundation demands under a yielding structure, detailing similar to the beams of the Special Moment Frame is not required. This "strong grade beam" provision could apply to both cantilever column systems and frame systems with the objective of avoiding the inelastic response or plastic hinging of the grade beam where it would be difficult to detect and repair after being subjected to strong ground motions.

Anchorage of the pile to the pile cap should be designed to permit energy dissipating mechanisms, such as pile slip at the pile-soil-interface, while maintaining a competent connection to the pile cap. A "least" capacity design approach is used for this purpose based on the pile section strength, not to exceed the load combination with overstrength, which simulates expected foundation demands under a yielding structure. A similar approach is also used for pile splice design.

Provisions are given to establish requirements as to when different pile analysis methods should be used. Short piles and long slender piles embedded in the earth behave differently when subject to lateral forces and displacements. Long slender pile response depends on its interaction with the soil considering the non-linear response of the soil. Long slender piles should be analyzed for lateral loads considering the nonlinear interaction of the shaft and soil. The nonlinearity is typically considered in the soil and not the pile. Numerous design aid curves and computer programs are available for this type of analysis, and such an analysis is not uncommon in practice (e.g. Ensoft, 2000a). This type of analysis is necessary to obtain realistic pile moments, forces and deflections. More sophisticated analyses which also consider nonlinear behavior or plastic hinging in the pile itself as well as nonlinearity in the soil for actual earthquake ground motions is still in the research realm. For pile length-to-diameter ratios less than or equal to 6, the pile can be treated as a rigid body simplifying the analysis. A method assuming a rigid body and linear soil response for lateral bearing is in current building codes. A more accurate and comprehensive approach using this method is given in a study by Czerniak (1957).

The effects of groups of piles, where closely spaced, should be taken into account for the soil vertical and horizontal response. As groups of closely spaced piles move laterally, failure zones for individual piles overlap and horizontal strength and stiffness response of the pile-soil system is reduced. Reduction factors or "*p-multipliers*" are needed to account for these groups of closely spaced piles. For a pile center-to-center spacing of three pile diameters, reduction factors of 0.6 for the leading pile row and 0.4 for the trailing pile rows are recommended by Rollins et al. (1999). Computer programs are available to analyze group effects, (e.g. Ensoft, 2000b).

Batter pile systems that are partially embedded have historically performed poorly under strong ground motions (Gerwick and Fotinos, 1992). Failure of battered piles has been attributed to neglecting the potential loading on the piles from ground deformation and also to an erroneous assumption that the lateral loads will be resisted by the axial response of piles leading to neglect of the induced moments in the pile at the pile cap (Lam and Bertero, 1990). Difficulties in examining fully embedded batter piles have led to uncertainties of the extent of damage for this type of foundation. Batter piles are considered as limited ductile systems by their nature and should be designed using the load combination with overstrength. Due to eccentricities inherent in batter pile configurations, moment resisting connections to the pile cap are required to resolve the statics. Otherwise the superstructure will have to resolve the eccentricities by resisting moments induced by the foundation under lateral forces. This concept is clearly illustrated in EQE Engineering (1991).

7.5.4.1 Uncased concrete piles. The uncased concrete piles category has been expanded to include auger-cast piles which are now subject to the same reinforcement requirements as other cast-in-place concrete piles. The longitudinal reinforcement within a pile has prescriptive termination point minimums intended for firm soils and is required to extend at least the full flexural length. The longitudinal reinforcement should extend past the flexural length by its development length requirement. The flexural length has been defined as that length of a pile from bottom of pile cap to a point where 0.4 times the concrete section cracking moment exceeds the calculated flexural demand. The 0.4 factor is analogous to a material resistance factor in strength design. This definition implies the plain concrete section will be resisting some minimal amount of moment demand beyond the "flexural" length. Where the pile is subject to significant uplift forces, it is recommended that the longitudinal reinforcement extend the full length of the pile.

Increased transverse reinforcement requirements are given for the potential plastic hinge zone immediately below the pile cap and for regions beyond that zone. The potential plastic hinge zone was taken to be three pile diameters below the pile cap to allow for the varied soil site classes from A through D. For soil site classes E and F, the potential plastic hinge zone is taken to be seven diameters in length as given in Section 7.5.4. The transverse reinforcement detailing for these zones is similar to that required for concrete Special Moment Frames at hinge regions and is expected to provide a displacement ductility of approximately 5 to 6 depending upon the axial load. However, recent studies and testing has substantiated that the soil will contribute substantially to the confinement of the concrete pile section in firm soils. Chai and Hutchison (1998) found that in-situ lateral load testing of 16 inch diameter conventionally reinforced circular piles performed to displacement ductilities from approximately 3 to 4 using a spiral steel reinforcement ratio of 0.0057. Further testing of the same piles but with a spiral steel reinforcement ratio of 0.0106 produced improved displacement ductilities and no spiral fracture failure which occurred in the prior piles tested with the lower spiral ratio. These circular spiral ratios are considerably less than those required by ACI 318 for columns in Seismic Design Category D. ACI 318 equation 10-6 would require a spiral reinforcement ratio of at least 0.0175 depending on the concrete core diameter. Budek, Benzoni and Priestley (1997) found that testing of 24 inch diameter circular conventionally reinforced concrete piles with a transverse (circular) reinforcement ratio of 0.006 offered adequate performance up to a displacement ductility of 4. Their conclusions indicate that the soil confinement can play a significant role in the pile shaft response As a result of these studies, full confinement reinforcement intended for superstructure columns is not necessary for in-ground pile foundations, except for soil site classes E and F and liquefiable soils. Beyond the potential plastic hinge region, the curvature ductility demand is not considered to exceed that provided by the nominal moment capacity of the section for nominal earthquake loads.

7.5.4.4 Precast-prestressed piles. The transverse reinforcement requirements are primarily taken from the PCI Committee Report (1993) on precast prestressed concrete piling for geographic regions subject to strong ground motions. The circular spiral transverse reinforcement equations recommended for precast prestressed concrete piles given in Park and Hoat Joen (1990) are the basis for the provisions. These equations make the curvature ductility capacity dependent on the pile axial load. A reduction of 50% in the normally required circular spiral reinforcement from those equations (similar to ACI 318 equation 10-6) was sufficient to achieve a displacement ductility of 4. The reduced circular spiral transverse reinforcement requirement is the basis for the PCI Piling Committee’s final provisions.

High strength hard drawn wire with higher yield strengths is permitted to be used for transverse circular spiral reinforcement of precast prestressed concrete piles. Pile test specimens using this type of transverse reinforcement include the research done by Park and Hoat Joen (1990). High strength hard drawn wire has yield strengths between 150 and 200 ksi. f_{yh} is conservatively limited to 85 ksi for this steel because hard drawn wire has limited ductililty.

7.5.4.5 Steel Piles. AISC Seismic (2002), Part I, Section 8.6 provides seismic design and detailing provisions for steel H-piles which should be used in conjunction with these provisions.

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Appendix to Chapter 7

GEOTECHNICAL ULTIMATE STRENGTH DESIGN OF FOUNDATIONS AND FOUNDATION LOAD-DEFORMATION MODELING

A7.2 GENERAL DESIGN REQUIREMENTS

A7.2.2. Foundation load capacities. In current geotechnical engineering practice, foundation design is based on allowable stresses, with allowable foundation load capacities, Q_{as}, for dead plus live loads based on limiting static settlements and providing a large factor of safety against exceeding ultimate capacities. In current practice, allowable soil stresses for dead plus live loads are increased by one-third for load combinations that include wind or seismic forces. The one-third increase is overly conservative if the allowable stresses for dead plus live loads are far below ultimate soil capacity.

This appendix provides guidance for the direct use of ultimate foundation load capacity, Q_{us}, for load combinations including seismic effects. It is required that foundations be capable of resisting loads with acceptable deformations considering the short duration of seismic loading, the dynamic properties of the soil, and the ultimate load capacities, Q_{us}, of the foundations under vertical, lateral, and rocking loading.

A7.2.2.1. Determination of ultimate foundation load capacities. For competent soils that are not expected to degrade in strength during seismic loading (e.g., due to partial or total liquefaction of cohesionless soils or strength reduction of sensitive clays), use of static soil strengths is recommended for determining ultimate foundation load capacities, Q_{us}. Use of static strengths is somewhat conservative for such soils because rate-of-loading effects tend to increase soil strengths for transient loading. Such rate effects are neglected because they may not result in significant strength increase for some soil types and are difficult to confidently estimate without special dynamic testing programs. The assessment of the potential for soil liquefaction or other mechanisms for reducing soil strengths is critical, because these effects may reduce soil strengths greatly below static strengths in susceptible soils.

The best-estimated ultimate vertical load capacity of footings, Q_{us}, should be determined using accepted foundation engineering practice. In the absence of moment loading, the ultimate vertical load capacity of a rectangular footing of width B and length L may be written as

$$Q_{us} = q_c BL$$

where q_c = ultimate soil bearing pressure.

For rigid footings subject to moment and vertical load, contact stresses become concentrated at footing edges, particularly as footing uplift occurs. The ultimate moment capacity, M_{us}, of the footing as limited by the soil is dependent upon the ratio of the vertical load stress, q, to the ultimate soil bearing pressure q_c. Assuming that contact stresses are proportional to vertical displacements and remain elastic up to q_c, it can be shown that uplift will occur prior to plastic yielding of the soils when q/q_c is less than 0.5. If q/q_c is greater than 0.5, then the soil at the toe will yield prior to uplift. This is illustrated in Figure CA7.2.2-1. In general the ultimate moment capacity of a rectangular footing may be expressed as:

$$M_{us} = \frac{LP}{2}\left(1 - \frac{q}{q_c}\right)$$

where

P = Vertical Load

$$q = \frac{P}{BL}$$

B = Footing width, and

L = Footing length in direction of rotation

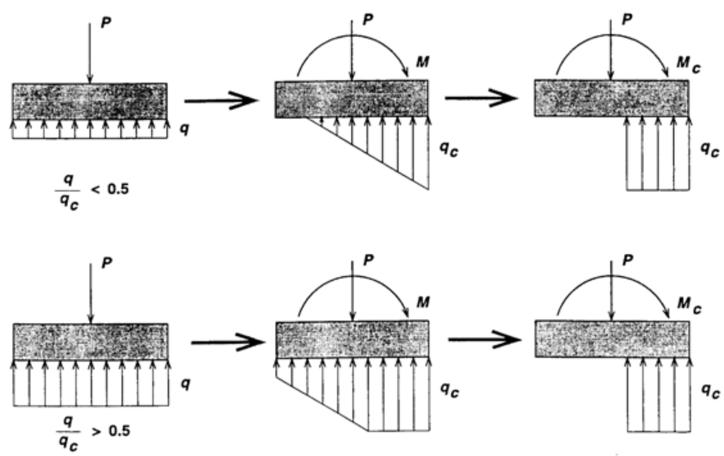


Figure CA7.2.2-1.

The ultimate lateral load capacity of a footing may be assumed equal to the sum of the best-estimated ultimate soil passive resistance against the vertical face of the footing plus the best-estimated ultimate soil friction force on the footing base. The determination of ultimate passive resistance should consider the potential contribution of friction on the face of the footing on the passive resistance.

For piles, the best-estimated ultimate vertical load capacity (for both axial compression and axial tensile loading) should be determined using accepted foundation engineering practice. When evaluating axial tensile load capacity, consideration should be given to the capability of pile cap and splice connections to take tensile loads.

The ultimate moment capacity of a pile group should be determined assuming a rigid pile cap, leading to an initial triangular distribution of axial pile loading from applied overturning moments. However, full axial capacity of piles may be mobilized when computing ultimate moment capacity, in a manner analogous to that described for a footing in Figure CA7.2.2-1. The ultimate lateral capacity of a pile group may be assumed equal to the best-estimated ultimate passive resistance acting against the edge of the pile cap and the additional passive resistance provided by piles.

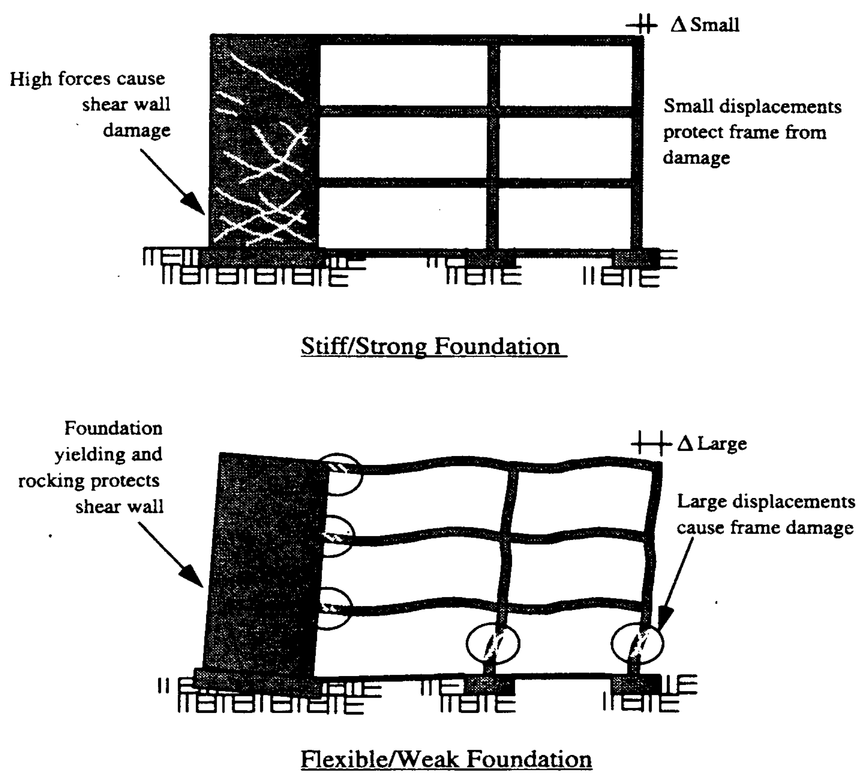
Resistance factors, ϕ, are provided to factor ultimate foundation load capacities, Q_{us}, to reduced capacities, ϕQ_{us}, used to check foundation acceptance criteria. The values of ϕ recommended in the *Provisions* are higher than those recommended in some codes and specifications for long-term static loading. The development of resistance factors for static loading has been based on detailed reliability studies and on calibrations to give designs and factors of safety comparable to those given by allowable stress design. As indicated in the first paragraph of this section, mobilized strengths for seismic loading conditions are expected to be somewhat higher than the static strengths specified for use in obtaining values of Q_{us}, especially for cohesive soils. In the absence of any detailed reliability studies for seismic loading conditions, Values of ϕ equal to 0.8 and 0.7 were selected for cohesive and cohesionless soils, respectively, where geotechnical site investigations, including laboratory or insitu tests, are conducted, and values of ϕ equal to 1.0 and 0.9 were selected where full-scale field testing of prototype foundations are conducted. These values are comparable to the values of 0.8 (for soil strengths determined based on a comprehensive site soil investigation including soil sampling and testing) and 0.9 (for soil strengths determined by site loading testing using plate bearing or near full scale foundation element testing) recommended by the SEAOC Seismology Committee Ad Hoc Foundation Committee (2001).

A7.2.2.2 Acceptance criteria. The factored load capacity, ϕQ_{us}, provides the basis for the acceptance criteria, particularly for the linear analysis procedures. The mobilization of ultimate capacity in the nonlinear analysis procedures does not necessarily mean unacceptable performance as structural deformations due to foundation displacements may be tolerable, as discussed by Martin and Lam (2000). For the nonlinear analysis procedures, it is also prudent to evaluate structural behavior utilizing parametric increases in foundation load capacities above Q_{us} by a factor of $1/\phi$, to check potential changes in structural ductility demands.

A7.2.3 Foundation load-deformation modeling. Analysis methods described in Sec. 5.3 (response spectrum procedure) and Sec. 5.4 (linear response history procedure), permit the use of realistic assumptions for foundation stiffness, as opposed to the assumption of a fixed base. In addition, the nonlinear response history procedure (Sec. 5.5) and the nonlinear static procedure (Appendix to Chapter 5) permit the use of realistic assumptions for the stiffness and load-carrying characteristics of the foundations. Guidance for flexible foundation (non-fixed base) modeling for the above analysis procedures are described herein.

Foundation load-deformation behavior characterized by stiffness and load capacity may significantly influence the seismic performance of a structure, with respect to both load demands and distribution among structural elements (ATC 1996, NEHRP 1997a, 1997b). This is illustrated schematically in Figure CA 7.2.3-1. While it is recognized that the load-deformation behavior of foundations is nonlinear, an equivalent elasto-plastic representation of load-deformation behavior is often assumed, as illustrated in Figure CA 7.2.3-2. To allow for variability and uncertainty in the selection of soil parameters and analysis methods used to determine stiffness and capacity, a range of parameters for foundation modeling should be used to permit sensitivity evaluations.

Foundation stiffness and strength affect various structural components differently.



Stiff/strong is not always favorable; nor is flexible/weak always conservative.

Figure CA7.2.3-1.

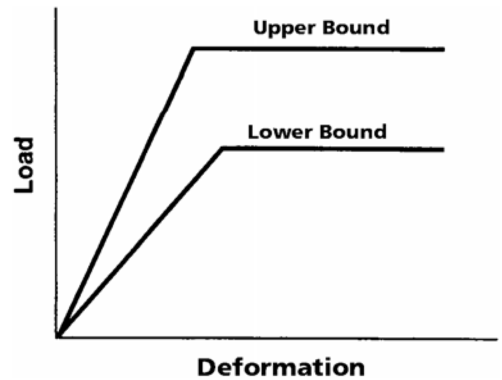


Figure CA7.2.3-2.

Consider the spread footing shown in Figure CA 7.2.3-3 with an applied vertical load (*P*), lateral load (*H*), and moment (*M*). The soil characteristics might be modeled as two translational springs and a rotational spring, each characterized by a linear elastic-stiffness and a plastic capacity. The use of a Winkler spring model acting in conjunction with the foundation to eliminate the rotational spring may also be used, as shown in Figure CA7.2.3-4. The Winkler model can capture more accurately progressive mobilization of plastic capacity during rocking behavior. Note the lateral action is normally uncoupled from the vertical and rotational action. Many foundation systems are relatively stiff and strong in the horizontal direction, due to passive resistance against the face of footings or basement walls, and friction beneath footings and floor slabs. Comparisons of horizontal stiffness of the foundation and the structure can provide guidance on the need to include horizontal foundation stiffness in demand or capacity analyses. In general, foundation rocking has the most influence on structural response. Slender shear wall structures founded on strip footings, in particular, are most sensitive to the effects of foundation rocking.

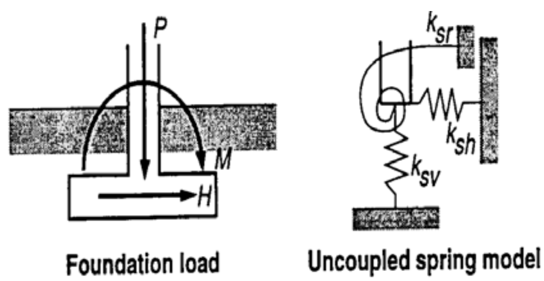


Figure CA7.2.3-3.

Assuming a shallow footing foundation may be represented by an embedded rigid plate in an elastic half-space, classical elastic solutions may be used to compute the uncoupled elastic stiffness parameters. Representative solutions are described in *Commentary* to Sec. 5.6. Solutions developed by Gazetas (1991) are also often used, as described in ATC (1996). Dynamic soil properties (i.e. properties consistent with seismic wave velocities and associated moduli of the soils as opposed to static soil moduli) should be used in dynamic soil solutions. The effects of nonlinearity on dynamic soil properties should be incorporated using the reduction factors in Sec. 5.6.2.1.1 or based on a site-specific study.

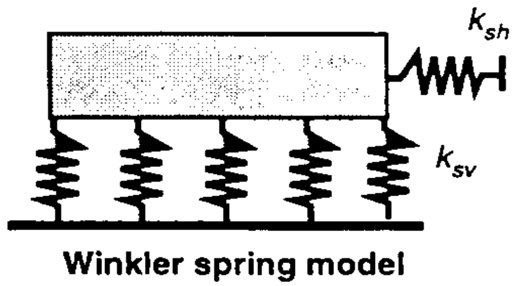


Figure CA 7.2.3-4.

In the case of pile groups, the uncoupled spring model shown in Figure CA 7.2.3-3 may also be used, where the footing represents the pile cap. In the case of the vertical and rotational springs, it can be assumed that the contribution of the pile cap is relatively small compared to the contribution of the piles. In general, mobilization of passive pressures by either the pile caps or basement walls will control lateral spring stiffness. Hence, estimates of lateral spring stiffness can be computed using elastic solutions as for footings. In instances where piles may contribute significantly to lateral stiffness (i.e., very soft soils, battered piles), solutions using beam-column pile models are recommended.

Axial pile group stiffness spring values, k_{sv}, are generally in the range given by:

$$k_{sv} = \sum_{n=1}^{N} \frac{0.5AE}{L} \text{ to } \sum_{n=1}^{N} \frac{2AE}{L}$$

where

A = Cross-sectional area of a pile,

E = Modulus of elasticity of piles,

L = Length of piles , and

N = Number of piles in group.

Values of axial stiffness depend on complex nonlinear interaction of the pile and soil (NEHRP, 1997b). For simplicity, best estimate values of *AE/L* and 1.5 *AE/L* are recommended for piles where axial capacity is primarily controlled by end bearing and side friction, respectively.

The rocking spring stiffness values, k_{sr}, about each horizontal pile cap axis may be computed by assuming each axial pile spring acts as a discrete Winkler spring. The rotational spring constant (moment per unit rotation) is then given by:

$$k_{sr} = \sum_{n=1}^{N} k_{vn} S_n^2$$

where

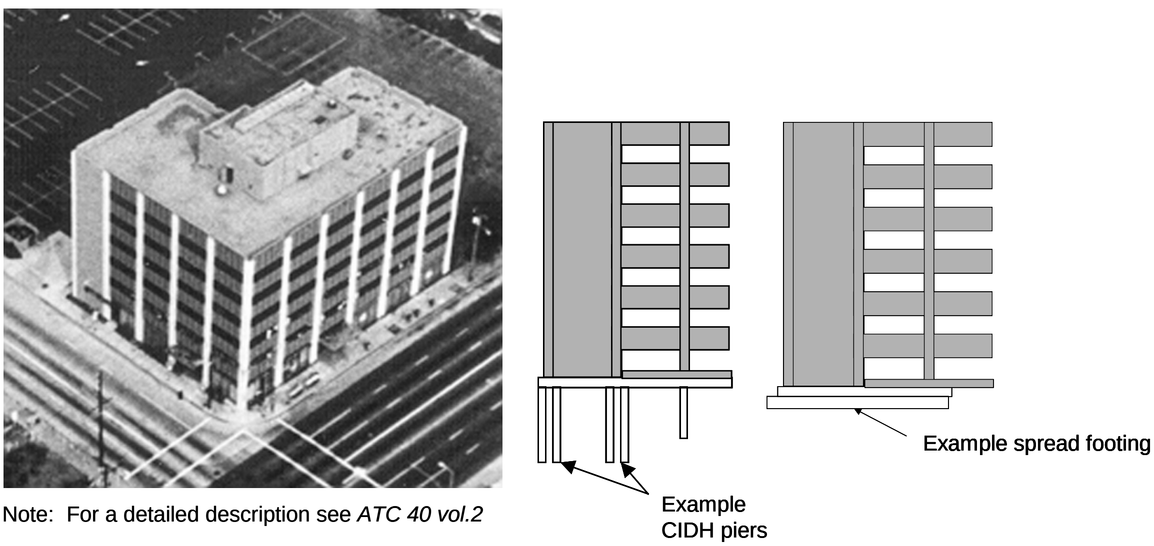
k_{vn} = Axial stiffness of the nth pile, and

S_n = Distance between nth pile and axis of rotation.

The effects of group action and the influence of pile batter are not accounted for in the above equations. These effects should be evaluated if judged significant.

Design Examples. In order to study and illustrate the effects of the change from allowable stress to ultimate strength design of foundations a series of examples were generated. The examples compared the size of foundations resulting from ultimate strength designs (USD) according the new procedures with those that would be obtained from conventional allowable stress designs (ASD).

The examples were based upon a single six-story reinforced concrete building with shear walls and gravity frame (see Figure 7.2.3-5). One set of examples was for a shallow spread footing design beneath a shear wall. The other set applied to deep CIDH piers placed beneath the same wall. For each set of examples, individual designs reflected a range of soil strengths and ASD factors of safety. The vertical loads were not changed, but two levels of seismic overturning demand were imposed.



Note: For a detailed description see *ATC 40 vol.2*

Figure 7.2.3-5 Example building.

While is not possible to generalize the results of these examples to apply universally, they are representative of the effects of the change to USD for a realistic case study. For the spread footing foundation the area of the footing for USD compared to that for ASD is controlled by the factor of safety applied to the soil strength for vertical loads. This reduction ranged from 0% to 20 percent for a low FOS (2) up to 25 percent to 40 percent for a high FOS (4). This is not surprising; where ASD uses a high factor of safety and is thus most conservative, USD results in a smaller footing size. However, the footing size cannot be smaller than that required for allowable stresses for static design under vertical dead plus live loads. For the pier example, the required length for USD was actually about 50 percent greater than for ASD for a low FOS (1.5) and up to 40 percent less for a high FOS (4).

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ATC, 1996, ATC 40, *Seismic Evaluation and Retrofit of Concrete Buildings*, 2 Volumes, The Applied Technology Council, Redwood City, CA.

Gazetas, G. (1991), “Foundation Vibrations,” *Foundation Engineering Handbook*, 2nd Edition, Vas Nostrand Reinhold, Edited by Hsai-Yang Fang.

Martin, G.R. and Lam, I.P., 2000, “Earthquake Resistant Design of Foundations: Retrofit of Existing Foundations,” *Proceedings*, GeoEng 2000, International Conference on Geological and Geotechnical Engineering, Melbourne, Australia, November.

NEHRP, 1997a, and 1997b, *National Earthquake Hazard Reduction Program Guidelines and Commentary for the Seismic Rehabilitation of Buildings*, developed for the Building Seismic Safety Council for the Federal Emergency Management Agency, (Publications FEMA 273 and 274).

SEAOC Seismology Committee, Ad Hoc Foundations Committee, 2001, “USD/LRFD/Limit State Approach to Foundation Design”, *Proceedings* of the 70th Annual SEAOC Convention, San Diego, California.

Chapter 8 Commentary

STEEL STRUCTURE DESIGN REQUIREMENTS

8.1 GENERAL

8.1.2 References. 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).

Revise the AISC Seismic Commentary Sec. C9.3 as follows: At the end of the second paragraph add the following: "This provision requires that the panel zone be proportioned using the method used to proportion the panel zone thickness of successfully tested connections. This should not be construed to mean that the thickness is required to be the same as the tested connection, only that the same method must be used to proportion it. For example, if the test were performed on a one-sided connection and the same beam and column sizes were used in a two-sided connection, the panel zone would be twice as thick as that of the tested connection."

8.2 GENERAL DESIGN REQUIREMENTS

8.2.1 Seismic Design Categories B and C. Structures assigned to Seismic Design Categories B and C do not require the same level of ductility capacity to provide the required performance as those assigned to the higher categories. For this reason, such structures are permitted to be designed using the requirements of any of the listed references, provided that the lower *R* value specified in Table 4.3-1 is used. Should the registered design professional choose to use the higher *R* values in the table, the detailing requirements for the higher Seismic Design Categories must be used.

8.2.2 Seismic Design Categories D, E, and F. Structures assigned to these categories must be designed in anticipation of significant ductility demands that may be placed on the structures during their useful life. Therefore, structures in these categories are required to be designed to meet special detailing requirements as referenced in this section.

8.4 COLD-FORMED STEEL

The allowable stress and allowable load levels in AISI are incompatible with the force levels calculated in accordance with these *Provisions*. It is therefore necessary to modify the provisions of AISI for use with the *Provisions*. ANSI/ASCE 8 and SJI are both based on LRFD and thus are consistent with the force levels in the *Provisions*. As such, only minor modifications are needed to correlate those load factors for seismic loads to be consistent with the *Provisions*. The modifications of all of the reference documents affect only designs involving seismic loads.

8.4.2 Light-frame walls. The provisions of this section apply to buildings framed with cold-formed steel studs and joists. Lateral resistance is typically provided by diagonal bracing (braced frames) or wall sheathing material. This section is only required for use in Seismic Design Categories D, E, and F. 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.

Table 8.4-1 presents nominal shear values for plywood and oriented strand board attached to steel stud wall assemblies. Design values are determined by multiplying the nominal values by a phi (ϕ) factor as presented in Sec. 8.4.2.5. These nominal values are based upon tests performed at Santa Clara University (Serrette, 1996). The test program included both cyclic and static tests; however, the values presented in Table 8.4-1 are based upon the cyclic tests as they are intended for use in seismic resistance. In low seismic areas where wind loads dominate, nominal values have been recommended for wind resistance by AISI based upon monotonic tests (Serrette, 1996). The cyclic tests were performed using the assemblies that static testing showed to be the most critical. The cyclically tested assemblies consisted of 3.5 by 1.625 in. C studs fabricated with ASTM A446 Grade A (33 ksi) material with a minimum base metal thickness of 0.033 in. Since the tests were conducted, ASTM A446 Grade A has been redesignated ASTM A653 SQ Grade 33. The test panels were 4 ft wide and 8 ft high, the sheathing material was applied vertically to only a single side of the studs, and there was no sheathing or bracing applied to the other side.

The cyclic tests were performed using a sequential phase displacement protocol under development at the time of the test by an ad hoc Committee of the Structural Engineers Association of Southern California. Nominal values were conservatively established by taking the lowest load in the last set of stable hysteretic loops. It is expected that subsequent testing of steel stud shear wall assemblies will reduce or modify some of the restrictive limits currently proposed for the use of the system such as the nominal maximum thickness of the studs of 0.043 in., the maximum aspect ratio of 2:1, and the ability to use sheathing on both sides of the wall.

8.4.4 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 ANSI/ASCE 8.

8.5 STEEL CABLES

The allowable stress levels of steel cable structures specified in ASCE 19 are modified for seismic load effects.

8.6 RECOMMENDED PROVISIONS FOR BUCKLING-RESTRAINED BRACED FRAMES

8.6.3 Commentary on Buckling-Restrained braced Frames (BRBF)

8.6.3.1. Scope. Buckling-restrained braced frames are a special class of concentrically braced frames. Just as in Special Concentrically Braced Frames (SCBF), the centerlines of BRBF members that meet at a joint intersect at a point to form a complete vertical truss system that resists lateral forces. BRBF have more ductility and energy absorption than SCBFs because overall brace buckling, and its associated strength degradation, is precluded at forces and deformations corresponding to the design story drift. See AISC Sections 13 and 14 for the effects of buckling in SCBF. AISC Seismic Figure C13.1 shows possible BRBF bracing configurations; note that neither x-bracing nor k-bracing is an option for BRBF.

BRBF are characterized by the ability of bracing elements to yield inelastically in compression as well as in tension. In BRBF the bracing elements dissipate energy through stable tension-compression yield cycles (Clark et. al, 1999). Figure C8.6.3.2 shows the characteristic hysteretic behavior for this type of brace as compared to that of a buckling brace. This behavior is achieved through limiting buckling of the steel core within the bracing elements. Axial stress is de-coupled from flexural buckling resistance; axial load is confined to the steel core while the buckling restraining mechanism, typically a casing, resists overall brace buckling and restrains high-mode steel core buckling (rippling).

Buckling-restrained braced frames are composed of columns, beams, and bracing elements, all of which are subjected primarily to axial forces. Braces of BRBF are composed of a steel core and a buckling-restraining system encasing the steel core. Figure C8.6.3.1 shows a schematic of BRBF bracing element (adapted from Tremblay et al., 1999). More examples of BRBF bracing elements are found in Watanabe et al., 1988; Wada et. al., 1994; and Clark et al., 1999. The steel core within the bracing element is intended to be the primary source of energy dissipation. During a moderate to severe earthquake the steel core is expected to undergo significant inelastic deformations.

BRBF can provide elastic stiffness that is comparable to that of EBF or SCBF. Full-scale laboratory tests indicate that properly designed and detailed bracing elements of BRBF exhibit symmetrical and stable hysteretic behavior under tensile and compressive forces through significant inelastic deformations (Watanabe et. al, 1988; Wada et. al, 1998; Clark et. al, 1999; Tremblay et. al, 1999). The ductility and energy dissipation capability of BRBF is expected to be comparable to that of SMF and greater than that of SCBF. This high ductility is attained by limiting buckling of the steel core.

The axial yield strength of the core, P_{ysc}, can be defined without dependence on other variables. This ability to control P_{ysc} significantly reduces the adverse effects of relying on nominal yield strength values. Careful proportioning of braces throughout the building height can result in specification of required P_{ysc} values that meet all of the strength and drift requirements of the applicable building code.

These provisions are based on the use of brace designs qualified by testing. They are intended to ensure that braces are used only within their proven range of deformation capacity, and that yield and failure modes other than stable brace yielding are precluded at the maximum inelastic drifts corresponding to the design earthquake. For analyses performed using linear methods, the maximum inelastic drifts for this system are defined as those corresponding to 150 percent of the design story drift. For nonlinear time-history analyses, the maximum inelastic drifts can be taken directly from the analyses results. This approach is consistent with the linear analysis equations for design story drift in the *1997 Uniform Building Code* and the *2003 NEHRP Recommended Provisions*. It is also noted that the consequences of loss of connection stability due to the actual seismic displacements exceeding the calculated values may be severe; braces are therefore required to have a larger deformation capacity than directly indicated by linear static analysis.

These provisions have been written assuming that future editions of *NEHRP Recommended Provisions* and of national codes will define system coefficients and limits for BRBFs. The assumed values for the response modification coefficient, system over strength factor, are deflection amplification factor are 8, 2, and 5.5 respectively. Height limits matching those for eccentrically braced frames are also expected.

The design engineer utilizing these provisions is strongly encouraged to consider the effects of configuration and proportioning of braces on the potential formation of building yield mechanisms. It is also recommended that engineers refer to the following documents to gain further understanding of this system: Watanabe et al., 1988, Reina et al., 1997, Clark et al., 1999, Tremblay et al., 1999, and Kalyanaraman et al., 1998.

During the planning stages of either a subassemblage or uniaxial brace test, certain conditions may exist that cause the test specimen to deviate from the parameters established in the testing appendix. These conditions may include:

- Availability of beam, column, and brace sizes that reasonably match those to be used in the actual building frame
- Test set-up limitations in the laboratory
- Actuator and reaction-block capacity of the laboratory
- Transportation and field-erection constraints
- Actuator to subassemblage connection conditions that require reinforcement of test

specimen elements not reinforced in the actual building frame

In certain cases, both building official and qualified peer reviewer may deem such deviations acceptable. The cases in which such deviations are acceptable are project-specific by nature and, therefore, do not lend themselves to further description in this *Commentary*. For these specific cases, it is recommended that the engineer of record demonstrate that the following objectives are met:

- Reasonable relationship of scale
- Similar design methodology
- Adequate system strength
- Stable buckling-restraint of the steel core
- Adequate rotation capacity
- Adequate cumulative strain capacity

8.6.3.2. Bracing Members

8.6.3.2.1 Composition

8.6.3.2.1.1 Steel Core. The steel core is composed of a yielding segment and steel core projections; it may also contain transition segments between the projections and yielding segment. The area of the yielding segment of the steel core is expected to be sized so that its yield strength is fairly close to the demand calculated from the applicable building code base shear. Designing braces close to the predicted required strengths will help ensure distribution of yielding over multiple stories in the building. Conversely, over-designing some braces more than others (e.g., by using the same size brace on all floors), may result in an undesirable concentration of inelastic deformations in only a few stories. The length and area of the yielding segment, in conjunction with the lengths and areas of the non-yielding segments, determine the stiffness of the brace. The yielding segment length and brace inclination also determines the strain demand corresponding to the design story drift.

In typical brace designs, a projection of the steel core beyond its casing is necessary in order to accomplish a connection to the frame. Buckling of this unrestrained zone is an undesirable yield mode and must therefore be precluded.

In typical practice, the designer specifies the core plate dimensions as well as the steel material and grade. The steel stress-strain characteristics may vary significantly within the range permitted by the steel specification, potentially resulting in significant brace overstrength. This overstrength must be addressed in the design of connections as well as of frame beams and columns.

In order to reduce this source of overstrength, the designer may choose to specify a brace capacity corresponding to a defined displacement (typically 150 percent of the design story drift) in the design documents. In addition, the designer may specify a limited range of acceptable yield stress in order to more strictly define the permissible range of core plate area. The brace supplier may then select the final core plate dimensions to meet the capacity requirement using the mill certificate or the results of a coupon test. The designer should be aware that this approach may result in a deviation from the calculated brace axial stiffness. The maximum magnitude of the deviation is dependant on the range of acceptable material yield stress. Designers following this approach should consider the possible range of stiffness in the building analysis in order to adequately address both the building period and expected drift.

8.6.3.2.1.2 Buckling-Restraining System. This term describes those elements providing brace stability against overall buckling. This includes the casing as well as elements connecting the core. The adequacy of the buckling-restraining system must be demonstrated by testing.

8.6.3.2.2 Testing. Testing of braces is considered necessary for this system. The applicability of tests to the designed brace is defined in Sec. 8.6.3.7. Sec. 9.2a, which describes in general terms the applicability of tests to designs, applies to BRBF.

BRBF designs require reference to successful tests of a similarly-sized test specimen and of a brace subassemblage that includes rotational demands. The former is a uniaxial test intended to demonstrate adequate brace hysteretic behavior. The latter is intended to verify the general brace design concept and demonstrate that the rotations associated with frame deformations do not cause failure of the steel core projection, binding of the steel core to the casing, or otherwise compromise the brace hysteretic behavior. A single test may qualify as both a subassemblage and a brace test subject to the requirements of Sec. 8.6.3.7; for certain frame-type subassemblage tests, obtaining brace axial forces may prove difficult and separate brace tests may be necessary. A sample subassemblage test is shown in Figure C8.6.3.5 (from Tremblay, 1999).

Tests cited serve another function in the design of BRBF: the maximum forces that the brace can deliver to the system are determined from test results. Calculation of these maximum forces is necessary for connection design and for the design of beams in *V*- and inverted-*V* configurations (see Sec. 8.6.3.4.1.3). In order to permit a realistic design of these beams, two separate calculations are made. The compression-strength adjustment factor, β, accounts for the compression overstrength (with respect to tension strength) noted in buckling-restrained braces in recent testing (SIE, 1999). The tension strength adjustment factor, ω, accounts for material overstrength (R_y) and strain hardening. Figure C-8.3.6.3 shows a diagrammatic bilinear force-displacement relationship in which the compression strength adjustment factor β and the tension-strength adjustment factor ω are related to brace forces and nominal material yield strength. These quantities are defined as:

$$\beta = \frac{\beta\omega F_y A}{\omega F_y A} = \frac{P_{\max}}{T_{\max}}$$

$$\omega = \frac{\omega F_y A}{F_y A} = \frac{T_{\max}}{F_y A}$$

where P_{max} is the maximum compression force and T_{max} is the maximum tension force within deformations corresponding to 150 percent of the Design Story Drift (these deformations are defined as $1.5D_{bm}$ in the Appendix on testing). The acceptance criteria for testing require that values of β and ω be greater than or equal to 1.0 for buckling-restrained braces.

8.6.3.2.3 Quality Assurance. The design provisions for BRBF's are predicated on reliable brace performance. In order to assure this performance, a quality assurance plan is required. These measures are in addition to those covered in the code of standard practice and Sec. 18. Examples of measures that may provide quality assurance are:

- Special inspection of brace fabrication.
- Inspection may include confirmation of fabrication and alignment tolerances, as well as NDT methods for evaluation of the final product.
- Brace manufacturer's participation in a recognized quality certification program.
- Certification should include documentation that the manufacturer's quality assurance plan is in compliance with the requirements of the BRBF Provisions, the Seismic Provisions for Structural Steel Buildings, and the Code of Standard Practice. The manufacturing and quality control procedures should be equal to, or better, than those used to manufacture brace test specimens.

8.6.3.3 Bracing Connections. Bracing connections must not yield at force levels corresponding to the yielding of the steel core; they are therefore designed for the maximum force that can be expected from the brace. In the actual building frame, the use of slip-critical bolts designed at factored loads is encouraged (but not required) to greatly reduce the contribution of bolt slip to the total inelastic deformation in the brace. Because of the way bolt capacities are calibrated, the engineer should recognize that the bolts are going to slip at load demands 30 percent lower than published factored capacities. This slippage is not considered to be detrimental to behavior of the BRBF system and is consistent with the design approach found elsewhere in Sec. 7.2 of AISC Seismic. See also commentary on Sec. C7.2 of AISC Seismic. Bolt holes may be drilled or punched subject to the requirements of LRFD Specification Sec. M2.5.

8.6.3.4 Special Requirements Related to Bracing Configuration

8.6.3.4.1. In SCBF, *V*-bracing has been characterized by a change in deformation mode after one of the braces buckles. This is due to the negative post-buckling stiffness, as well as the difference between tension and compression capacity, of traditional braces. Since buckling-restrained braces do not exhibit the negative secant stiffness associated with post-buckling deformation, and have only a small difference between tension and compression capacity, the practical requirements of the design provisions for this configuration are relatively minor. Figure C8.6.3.4 shows the deformation mode that develops after one brace has yielded but before the yielding of the opposite brace completes the mechanism. This mode involves flexure of the beam and elastic axial deformation of the un-yielded brace; it also involves inelastic deformation of the yielded brace that is much greater than the elastic deformation of the opposing brace. The drift range that corresponds to this deformation mode depends on the flexural stiffness of the beam. Therefore, where V-braced frames are used, it is required that a beam be provided that has sufficient stiffness, as well as strength, to permit the yielding of both braces within a reasonable story drift considering the difference in tension and compression capacities determined by testing.

The beam is expected to undergo this deflection, which is permanent, during moderate seismic events; a limit is therefore applied to this deflection. Additionally, the required brace deformation capacity must include the additional deformation due to beam deflection under this load. Since other requirements such as the brace testing protocol (Sec. 8.6.3.7.6.3) and the stability of connections (Sec.8.6.3.3.3) depend on this deformation, engineers will find significant incentive to avoid flexible beams in this configuration. Where the special configurations shown in Figure C13.3 of AISC Seismic are used, the requirements of this section are not relevant.

8.6.3.5 Columns. Columns in BRBF are required to have compact sections because some inelastic rotation demands are possible. Columns are also required to be designed considering the maximum force that the adjoining braces are expected to develop.

8.6.3.6 Beams. Like columns, beams in BRBF are required to have compact sections because some inelastic rotation demands are possible. Likewise, they are also required to be designed considering the maximum force that the adjoining braces are expected to develop.

8.6.3.7.1 Scope and Purpose. Development of the testing requirements in these provisions was motivated by the relatively small amount of test data on this system available to structural engineers. In addition, no data from the response of BRBFs to severe ground motion is available. Therefore, the seismic performance of these systems is relatively unknown compared to more conventional steel-framed structures.

The behavior of a buckling restrained brace frame differs markedly from conventional braced frames and other structural steel seismic-force-resisting systems. Various factors affecting brace performance under earthquake loading are not well understood and the requirement for testing is intended to provide assurance that the braces will perform as required, and also to enhance the overall state of knowledge of these systems.

It is recognized that testing of brace specimens and subassemblages can be costly and time-consuming. Consequently, this Chapter has been written with the simplest testing requirements possible, while still providing reasonable assurance that prototype BRBFs based on brace specimens and subassemblages tested in accordance with these provisions will perform satisfactorily in an actual earthquake.

It is not intended that these provisions drive project-specific tests on a routine basis for building construction projects. In most cases, tests reported in the literature, or supplied by the brace manufacturer, can be used to demonstrate that a brace and subassemblage configuration satisfies the strength and inelastic rotation requirements of these provisions. Such tests, however, should satisfy the requirements of this Chapter.

The provisions have been written allowing submission of data on previously tested, based on similarity conditions. As the body of test data for each brace type grows, the need for additional testing is expected to diminish. The provisions allow for manufacturer-designed braces, through the use of the design methodology.

Most testing programs developed for primarily axial-load-carrying components focus largely on uniaxial testing. However, these provisions are intended to direct the primary focus of the program toward testing of a subassemblage that imposes combined axial and rotational deformations on the brace specimen. This reflects the view that the ability of the brace to accommodate the necessary rotational deformations cannot be reliably predicted by analytical means alone. Subassemblage test requirements are discussed more completely in Sec. 8.6.3.7.4.

Where conditions in the actual building differ significantly from the test conditions specified in this Chapter, additional testing beyond the requirements described herein may be needed to assure satisfactory brace performance. Prior to developing a test program, the appropriate regulatory agencies should be consulted to assure the test program meets all applicable requirements.

8.6.3.7.2 Symbols. The provisions require the introduction of several new variables. The quantity Δ_{bm} represents both an axial displacement and a rotational quantity. Both quantities are determined by examining the profile of the building at the design story drift, Δ_m, and extracting joint lateral and rotational deformation demands.

Determining the maximum rotation imposed on the braces used in the building may require significant effort. The engineer may prefer to select a reasonable value (i.e., interstory drift), which can be simply demonstrated to be conservative for each brace type, and is expected to be within the performance envelope of the braces selected for use on the project.

The brace deformation at first significant yield is used in developing the test sequence described in Sec. 8.6.3.7.6.3. The quantity is required to determine the actual cumulative inelastic deformation demands on the brace. If the nominal yield stress of the steel core were used to determine the test sequence, and significant material over-strength were to exist, the total inelastic deformation demand imposed during the test sequence would be overestimated.

8.6.3.7.3 Definitions. Two types of testing are referred to in this Chapter. The first type is subassemblage testing, described in Sec. 8.6.3.7.4, an example of which is illustrated in Figure C8.6.3.5.

The second type of testing described in Sec. 8.6.3.7.5 as brace specimen testing is permitted to be uniaxial testing.

8.6.3.7.4 Subassemblage Test Specimen. The objective of subassemblage testing is to verify the ability of the brace, and in particular its steel core extension and buckling restraining mechanism, to accommodate the combined axial and rotational deformation demands without failure.

It is recognized that subassemblage testing is more difficult and expensive than uniaxial testing of brace specimens. However, the complexity of the brace behavior due to the combined rotational and axial demands, and the relative lack of test data on the performance of these systems, indicates that subassemblage testing should be performed.

Subassemblage testing is not intended to be required for each project. Rather, it is expected that brace manufacturers will perform the tests for a reasonable range of axial loads, steel core configurations, and other parameters as required by the provisions. It is expected that this data will subsequently be available to engineers on other projects. Manufacturers are therefore encouraged to conduct tests that establish the device performance limits to minimize the need for subassemblage testing on projects.

Similarity requirements are given in terms of measured axial yield strength of both the prototype and the test specimen braces. This is better suited to manufacturer's product testing than to project-specific testing. Comparison of mill certificate or coupon test results is a way to establish a similarity between the Subassemblage Test Specimen brace and the Prototype braces. Once similarity is established, it is acceptable to fabricate Test Specimens and Prototype braces from different heats of steel.

A variety of subassemblage configurations are possible for imposing combined axial and rotational deformation demands on a test specimen. Some potential subassemblages are shown in Figure C8.3.6. The subassemblage need not include connecting beams and columns provided that the test apparatus duplicates, to a reasonable degree, the combined axial and rotational deformations expected at each end of the brace.

Rotational demands may be concentrated in the steel core extension in the region just outside the buckling restraining mechanism. Depending on the magnitude of the rotational demands, limited flexural yielding of the steel core extension may occur. Rotational demands can also be accommodated by other means, such as tolerance in the buckling restraint layer or mechanism, elastic flexibility of the brace and steel core extension, or through the use of pins or spherical bearing assemblies. It is in the engineer's best interest to include in a subassemblage testing all components that contribute significantly to accommodating rotational demands. The use of pins, while accommodating rotational demands, creates the potential for instability; and should be carefully considered by the engineer.

It is intended that the subassemblage test specimen be larger in axial-force capacity than the Prototype. However, the possibility exists for braces to be designed with very large axial forces. Should the brace yield force be so large as to make subassemblage testing impractical, the engineer is expected to make use of the provisions that allow for alternate testing programs, based on building official approval and qualified peer review. Such programs may include, but are not limited to, non-linear finite element analysis, partial specimen testing, and reduced-scale testing, in combination with full-scale uniaxial testing where applicable or required.

The steel core material was not included in the list of requirements. The more critical parameter, calculated margin of safety for the steel core projection stability, is required to meet or exceed the value used in the prototype. The method of calculating the steel core projection stability should be included in the design methodology.

8.6.3.7.5 Brace Test Specimen. The objective of brace test specimen testing is to establish basic design parameters for the BRBF system.

It is recognized that the fabrication tolerances used by brace manufacturers to achieve the required brace performance may be tighter than those used for other fabricated structural steel members. The engineer is cautioned against including excessively prescriptive brace specifications, as the intent of these provisions is that the fabrication and supply of the braces is achieved through a performance-based specification process. It is considered sufficient that the manufacture of the test specimen and the prototype braces be conducted using the same quality control and assurance procedures, and the braces be designed using the same design methodology.

The engineer should also recognize that manufacturer process improvements over time may result in some manufacturing and quality control and assurance procedures changing between the time of manufacture of the brace test specimen and of the prototype. In such cases reasonable judgment is required.

If the steel core or steel core projection is not biaxially symmetric, the engineer should ensure that the same orientation is maintained in both the test specimen and the prototype.

The allowance of previous test data (similarity) to satisfy these provisions is less restrictive for uniaxial testing than for subassemblage testing. Subassemblage test specimen requirements are described in Sec. C8.6.3.7.4.

A considerable number of uniaxial tests have been performed on some brace systems and the engineer is encouraged, wherever possible, to submit previous test data to meet these provisions. Relatively few Subassemblage tests have been performed. This type of testing is considered a more demanding test of the overall brace performance.

8.6.3.7.5.4 Connection Details. In many cases it will not be practical or reasonable to test the exact brace connections present in the prototype. These provisions are not intended to require such testing. In general, the demands on the steel core extension to gusset-plate connection are well defined due to the known axial capacity of the brace and the limited flexural capacity of the steel core extension. The subsequent design of the bolted or welded connection is relatively well-understood and it is not intended that these connections become the focus of the testing program.

For the purposes of utilizing previous test data to meet the requirements of this Chapter, the requirements for similarity between the brace and subassemblage brace test specimen can be considered to exclude the steel core extension connection to frame.

8.6.3.7.5.5 Materials. The intent of the provisions is to allow test data from previous test programs to be presented where possible. See Sec.8.6.3.7.4 for additional commentary.

8.6.3.7.5.7 Bolts. For the brace test specimen, it is crucial to treat the ultimate load that can be expected in the braces as the load at which bolt slippage should be prevented. Prevention of bolt slippage increases the chances of achieving a successful test and protects laboratory setup. In terms of the nomenclature used by the Research Council on Structural Connections (RCSC), prevention of bolt slippage implies using service-level load capacities when sizing bolted connections. Bolted connections sized using service-level capacities per RCSC will provide at least a 90 percent reliability that the bolts will not slip at the maximum force developed by the braces during the test.

The intent of this provision is to ensure that the bolted end-connections of the brace test specimen reasonably represent those of the prototype. It is possible that due to fabrication or assembly constraints variations in faying-surface preparation, bolt-hole fabrication, and bolt size may occur. In certain cases, such variations may not be detrimental to the qualification of a successful cyclic test. Final acceptability of variations in brace-end bolted connection rest on the opinion of the building official or qualified peer reviewer.

8.6.3.7.6.3 Loading Sequence. The subassemblage test specimen is required to undergo combined axial and rotational deformations similar to those in the prototype. It is recognized that identical braces, in different locations in the building, will undergo different maximum axial and rotational deformation demands. In addition, the maximum rotational and axial deformation demands may be different at each end of the brace. The engineer is expected to make simplifying assumptions to determine the most appropriate combination of rotational and axial deformation demands for the testing program.

Some subassemblage configurations will require that one deformation quantity be fixed while the other is varied as described in the test sequence above. In such a case, the rotational quantity may be applied and maintained at the maximum value, and the axial deformation applied according to the test sequence. The engineer may wish to perform subsequent tests on the same subassemblage specimen to bound the brace performance.

The loading sequence requires each tested brace to achieve ductilities corresponding to 1.5 times the design story drift and a cumulative inelastic axial ductility capacity of 140. Both of these requirements are based on a study in which a series nonlinear dynamic analyses was conducted on model buildings in order to investigate the performance of this system; the ductility capacity requirement represents a mean of response values and the cumulative ductility capacity requirement is a mean plus standard deviation value (Sabelli, 2001). In that study, buildings were designed and models of brace hysteresis selected so as to maximize the demands on braces. It is therefore believed that these requirements are more severe than the demands that typical braces in typical designs would face under their design-basis ground motion, perhaps substantially so. It is also expected that as more test data and building analysis results become available these requirements may be revisited.

The ratio of brace yield deformation (D_{by}) to the brace deformation corresponding to the design story drift (D_{bm}) must be calculated in order to define the testing protocol. This ratio is typically the same as the ratio of the displacement amplification factor (as defined in the applicable building code) to the actual overstrength of the brace; the minimum overstrength is defined in Sec. 8.6.3.2.1.1.1. Engineers should note that there is a minimum brace deformation demand corresponding to 1% story drift (8.6.3.7.2); provision of overstrength beyond that required to so limit the design story drift may not be used as a basis to reduce the testing protocol requirements.

Table C8.6.3.7.1 shows an example brace test protocol. For this example, it is assumed that the brace deformation corresponding to the design story drift is four times the yield deformation; it is also assumed that the design story drift is larger than the 1 percent minimum. The test protocol is then constructed from steps 1-4 of Sec. 8.6.3.7.6.3. In order to calculate the cumulative inelastic deformation, the cycles are converted from multiples of brace deformation at the design story drift (D_{bm}) to multiples of brace yield deformation (D_{by}). Since the cumulative inelastic drift at the end of the 1.5D_{bm} cycles is less than the minimum of 140D_{by} required for brace tests, additional cycles to D_{bm} are required. At the end of three such cycles, the required cumulative inelastic deformation has been reached.

Table C8.6.3.7.1 Example Brace Testing Protocol

Cycle Deformation	Deformation Inelastic Deformation	Inelastic	Cumulative
6 @ D_{by}= 0D_{by}	= 6*4*($D_{by} - D_{by}$)	= 0D_{by}	0D_{by}
4 @ 0.5D_{bm}= 16D_{by}	= 4 @ 2.0D_{by}	= 4*4*(2.0$D_{by} - D_{by}$)	= 16D_{by} 0D_{by} +16D_{by}
4 @ D_{bm}= 64D_{by}	= 4 @ 4.0D_{by}	= 4*4*(4.0$D_{by} - D_{by}$)	= 48D_{by} 16D_{by} +48D_{by}
2 @ 1.5D_{bm}= 104D_{by}	= 2 @ 6.0D_{by}	= 2*4*(6.0$D_{by} - D_{by}$)	= 40D_{by} 64D_{by} +40D_{by}
3 @ D_{bm}= 140D_{by}	= 3 @ 4.0D_{by}	= 3*4*(4.0$D_{by} - D_{by}$)	= 36D_{by} 104D_{by} +36D_{by}

Cumulative Inelastic Deformation at End of Protocol = 140 D_{by}

Dynamically applied loads are not required by these provisions. The use of slowly applied cyclic loads, widely described in the literature for brace specimen tests, is acceptable for the purposes of these provisions. It is recognized that dynamic loading can considerably increase the cost of testing, and that few laboratory facilities have the capability to apply dynamic loads to very large-scale test specimens. Furthermore, the available research on dynamic loading effects on steel test specimens has not demonstrated a compelling need for such testing.

If rate-of-loading effects are thought to be potentially significant for the steel core material used in the prototype, it may be possible to estimate the expected change in behavior by performing coupon tests at low (test cyclic loads) and high (dynamic earthquake) load rates. The results from brace tests would then be factored accordingly.

8.6.3.7.8 Materials Testing Requirements. Tension testing of the steel core material used in the manufacture of the test specimens is required. In general, there has been good agreement between coupon test results and observed tensile yield strengths in full-scale uniaxial tests. Material testing required by this appendix is consistent with that required for testing of beam-to-column moment connections. For further information on this topic refer to Section CS8 of AISC Seismic.

8.6.3.7.10 Acceptance Criteria. The acceptance criteria are written so that the minimum testing data that must be submitted is at least one subassemblage test and at least one uniaxial test. In most cases the subassemblage test also qualifies as a uniaxial test provided the requirements of Sec. 8.6.3.7.5 are met. If project specific subassemblage testing is to be performed it may be simplest to perform two subassemblage tests to meet the requirements of this section. For the purposes of these requirements a single subassemblage test incorporating two braces in a chevron or other configuration is also considered acceptable.

Depending on the means used to connect the test specimen to the subassemblage or test apparatus, and the instrumentation system used, bolt slip may appear in the load vs. displacement history for some tests. This may appear as a series of spikes in the load vs. displacement plot and is not generally a cause for concern, provided the behavior does not adversely affect the performance of the brace or brace connection.

These acceptance criteria are intended to be minimum requirements. The 1.3 limit in Sec. 8.6.3.7.10.5 is essentially a limitation on β. These provisions were developed assuming that $\beta < 1.3$ so this provision has been included in the test requirements. Most currently available braces should be able to satisfy this requirement.

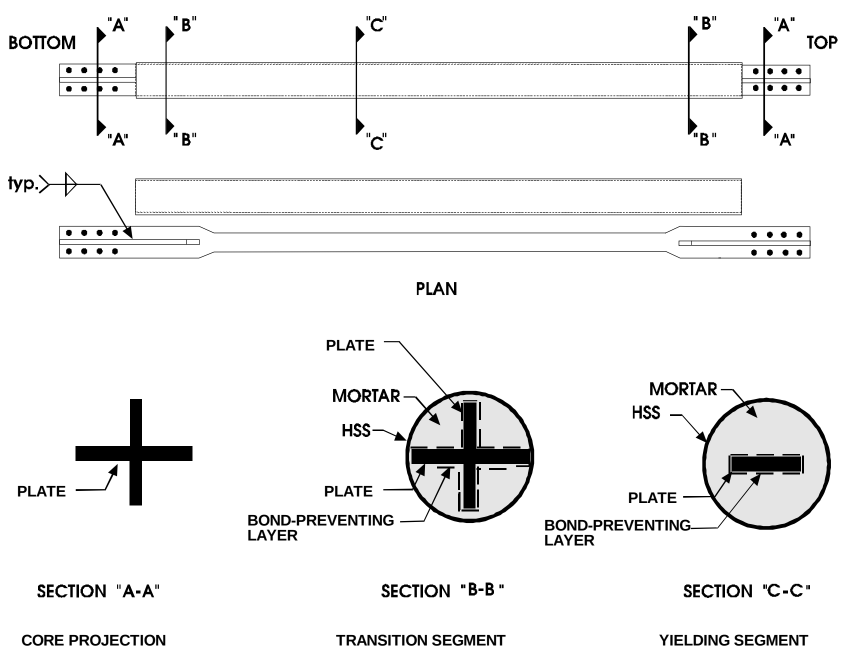

Figure C8.6.3.1 Detail of a Buckling Restrained Brace (Courtesy of R. Tremblay).

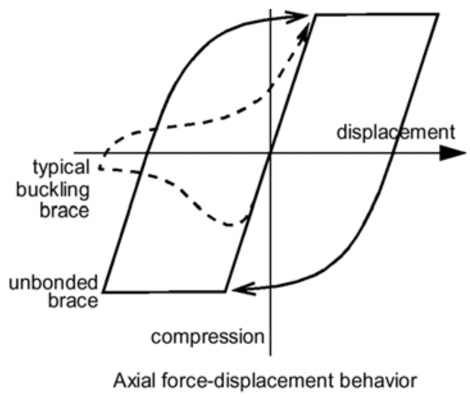

Figure 8.6.3.2 Detail of buckling-restrained (unbonded) brace hysteretic behavior (Courtesy of Seismic Isolation Engineering).

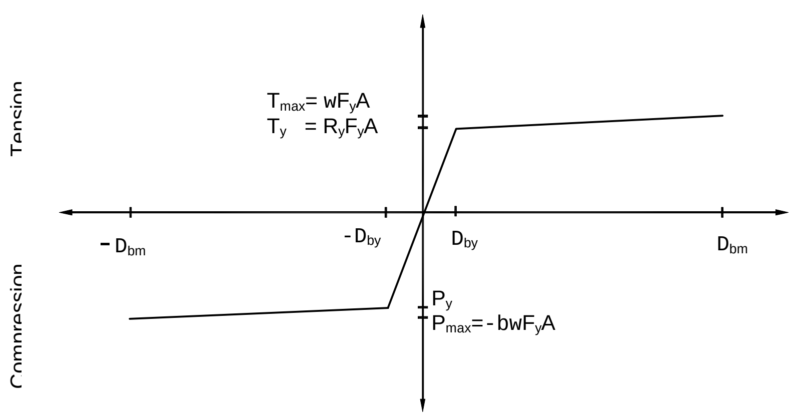

Figure C8.6.33 Diagram of brace force-displacement.

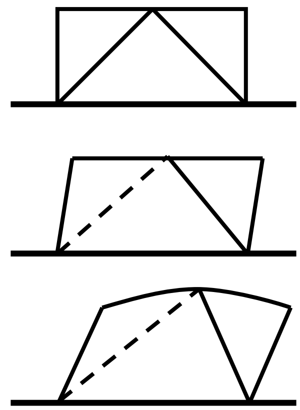

Figure C8.6.3.4 Post-yield, pre-mechanism change in deformation mode for V- and Inverted-V BRBF.

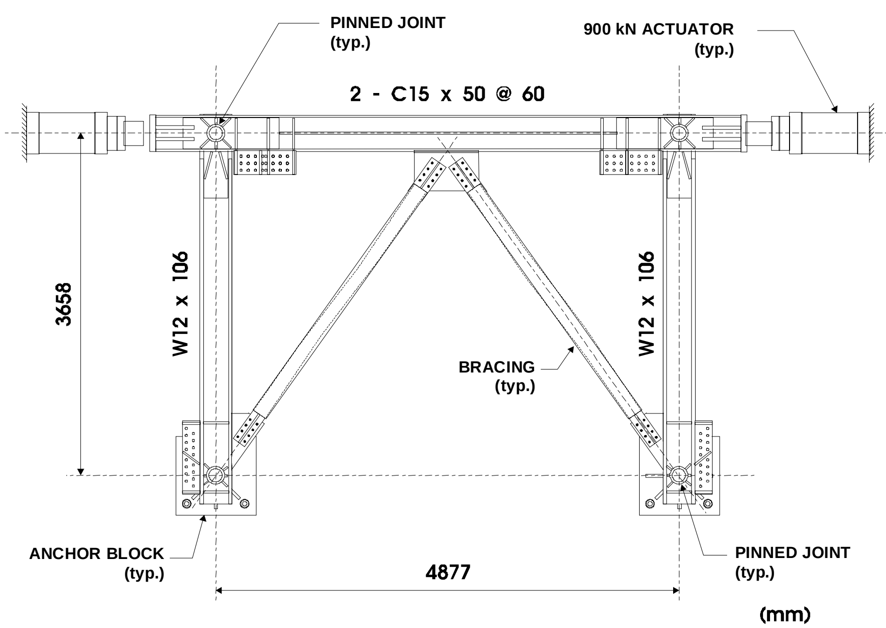

Figure C8.6.3.5 Example of test subassemblage.

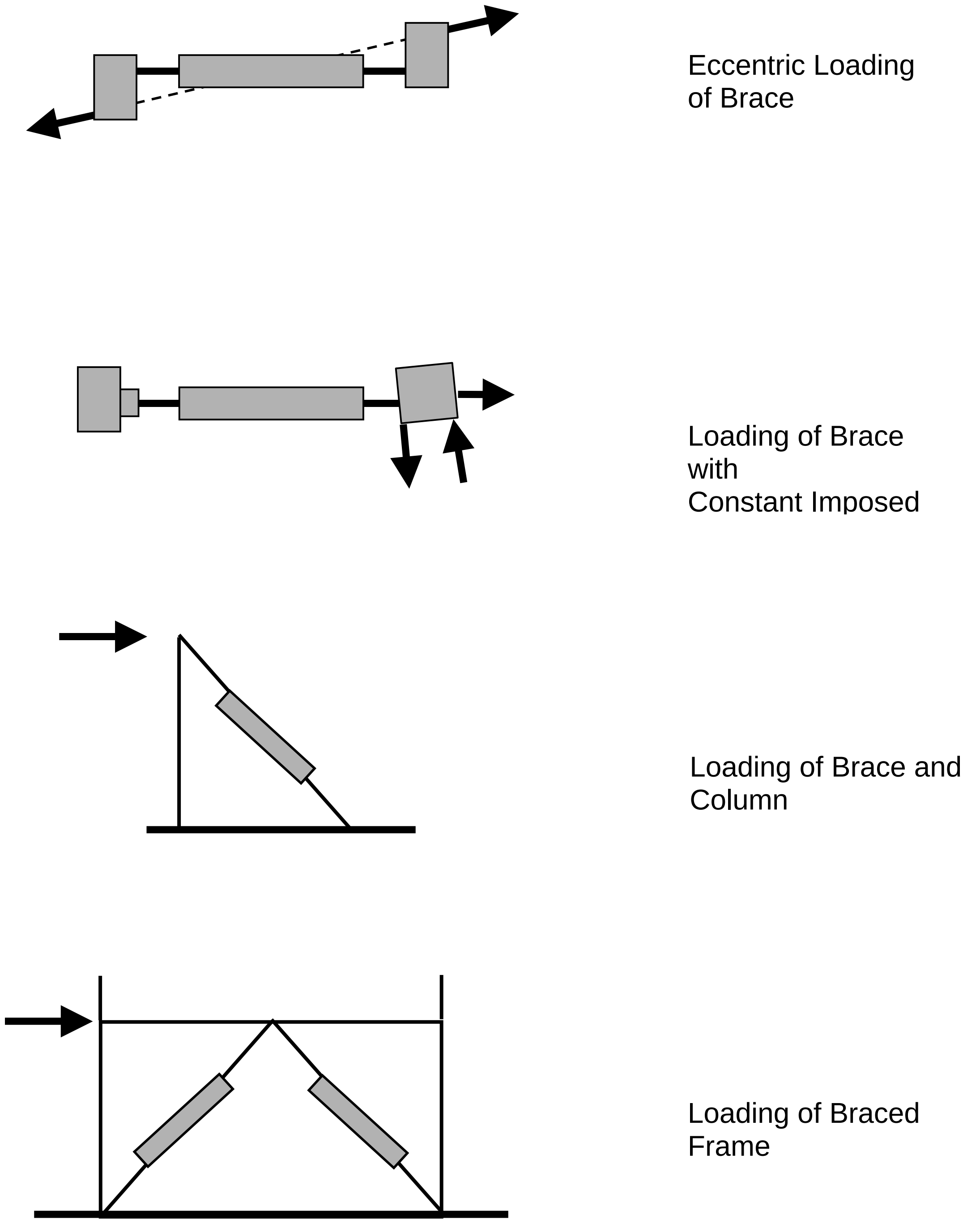

Figure C8.6.3.6 Schematic of possible test subassemblages.

8.7. SPECIAL STEEL PLATE WALLS

8.7.3 Scope. These provisions for SSPWs are intended for use in conjunction with AISC Seismic.

In special steel plate walls (SSPWs), the slender unstiffened steel plates (webs) connected to surrounding horizontal and vertical boundary elements (i.e., HBEs and VBEs) are designed to yield and behave in a ductile hysteretic manner during earthquakes. All HBEs are also rigidly connected to the VBEs with moment resisting connections able to develop the expected plastic moment of the HBEs. Each web must be surrounded by boundary elements.

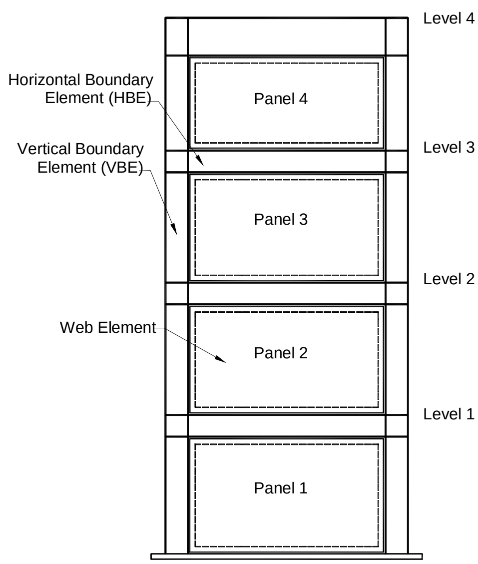

Figure C8.7.3.1 Schematic of Special Steel Plate Walls.

Experimental research on SSPWs subjected to cyclic inelastic quasi-static and dynamic loading (Thorburn, et al., 1983; Timler and Kulak, 1983; Tromposch and Kulak, 1987; Roberts and Sabouri-Ghomi, 1992; Cassese et al., 1993; Driver et al., 1997; Elgaaly 1998; Rezai, 1999; Lubell et al., 2000;) has demonstrated their ability to behave in a ductile manner and dissipate significant amounts of energy. This has been confirmed by analytical studies using finite element analysis and other analysis techniques (Sabouri-Ghomi and Roberts, 1992; Elgaaly et al., 1993; Elgaaly and Liu, 1997; Driver et al., 1997).

Yielding of the webs occurs by development of tension field action at an angle close to 45° from the vertical, and buckling of the plate in the orthogonal direction. Past research shows that the sizing of VBEs and HBEs in a SSPW makes it possible to develop this tension field action across the entire webs. Except for cases with very stiff HBEs and VBEs, yielding in the webs develops in a progressive manner across each panel. Because the webs do not yield in compression, continued yielding upon repeated cycles of loading is contingent upon the SSPW being subjected to progressively larger drifts, except for the contribution of plastic hinging developing in the HBEs to the total system hysteretic energy. In past research (Driver et al. 1997), the yielding of boundary elements contributed approximately 25-30 percent of the total load strength of the system.

With the exception of plastic hinging at the ends of HBEs, the surrounding horizontal and vertical boundary elements are designed to remain essentially elastic when the webs are fully yielded. Plastic hinging at the ends of HBEs is needed to develop the plastic collapse mechanism of this system. Plastic hinging in the middle of HBEs, which could partly prevent yielding of the webs, is deemed undesirable. Cases of both desirable and undesirable yielding in VBEs have been observed in past testing. In absence of a theoretical formulation to quantify the conditions leading to acceptable yielding (and supporting experimental validation of this formulation), the conservative requirement of elastic VBE response is justified.

Research literature often compares the behavior of steel plate walls to that of a vertical plate girder, indicating that the webs of a SSPW resist shears by tension field action (similarly to the webs of a plate girder) and that the VBEs of a SSPW resist overturning moments (similarly to the flanges of a plate girder). While this analogy is useful in providing a conceptual understanding of the behavior of SSPWs, many significant differences exist in the behavior and strength of the two systems. Past research shows that the use of structural shapes for the VBEs and HBEs in SSPWs (as well as other dimensions and details germane to SSPWs) favorably impacts orientation of the angle of development of the tension field action, and makes possible the use of very slender webs (having negligible diagonal compressive strength). Sizeable top and bottom HBEs are also required in SSPWs to anchor the significant tension fields that develop at these ends of the structural system. Limits imposed on the maximum web slenderness of plate girders to prevent flange buckling, or due to transportation requirements, are also not applicable to SSPWs which are constructed differently. For these reasons, the use of Appendix G in AISC LRFD Specifications for the design of SSPW is not appropriate.

8.7.4 Webs. The specified minimum yield stress of steel used for SSPW is per Sec. 6.1. However, the webs of SSPWs could also be of special highly ductile low yield steel having specified minimum yield in the range of 12 to 33 ksi.

8.7.4.1. The lateral shears are carried by tension fields that develop in the webs stressing in the direction α, defined in Sec. 8.7.4. To determine α, when the HBEs and VBEs boundary elements of a web are not identical, the average of HBE areas may be taken in the calculation of A_b, and the average of VBE areas and inertias may be respectively used in the calculation A_c, and I_c.

Plastic shear strength of panels is given by $0.5\ R_y\ F_y\ t_w\ L_{cf}\ \sin 2\ \alpha$. Nominal strength is obtained by dividing this value by a system overstrength, as defined by FEMA 369, and taken as 1.2 for SSPWs (Berman and Bruneau 2003).

The above plastic shear strength is obtained from the assumption that, for purposes of analysis, each web may be modeled by a series of inclined pin-ended strips Figure C8.7.4-1, oriented at angle α. Past research has shown this model to provide realistic results, as shown in Figure C8.7.4-2 for example, provided at least 10 equally spaced strips are used to model each panel.

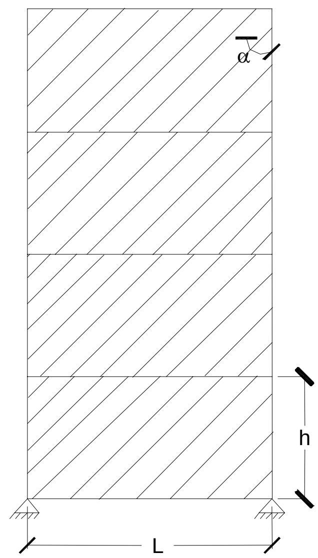

Figure C8.7.4-1 Strip model of a SSPW.

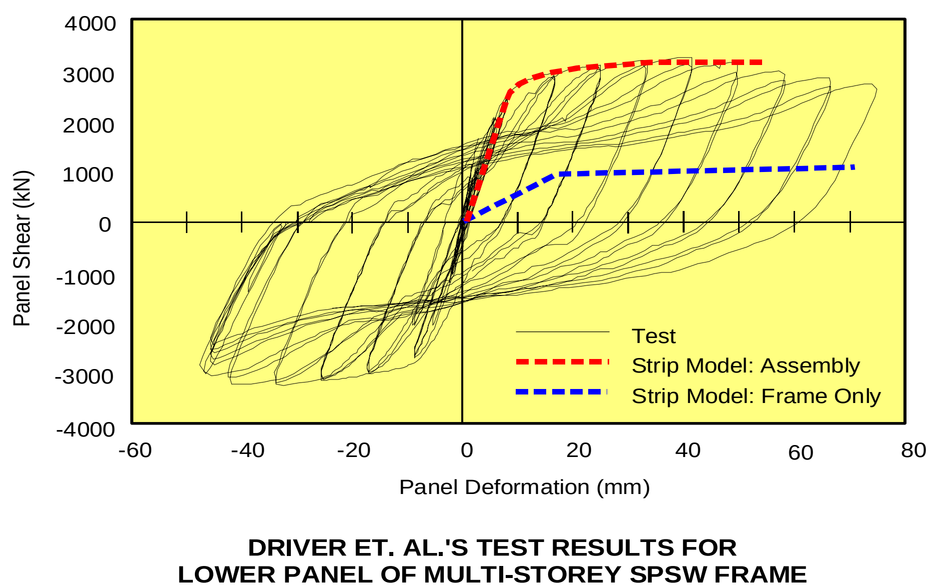

Figure C8.7.4-2 Comparison of Experimental Results and Strength predicted by Strip Model (Driver et al. 1997).

8.7.4.2 Panel Aspect Ratio. Past research shows that modeling SSPWs with strips is reasonably accurate for panel aspect ratios of *L/h* that exceed 0.8 (Rezai 1999). Additional horizontal intermediate boundary elements could be introduced in SSPW to modify the *L/h* of panels having such an aspect ratio less than 0.8.

No theoretical upper bound exists on *L/h* (provided sufficiently stiff HBEs can be provided), but a maximum value of 2.5 is specified on the basis that past research has not investigated the seismic behavior of SSPWs having *L/h* greater than 2.0. Excessive flexibility of HBEs is of concern for *L/h* ratio beyond the specified limit. For conditions beyond the specified limits, other FEM methods shall be used which correlate with published test data.

8.7.4.2 Openings in webs. Large openings in webs create significant local demands and thus must have HBE and VBE in a similar fashion as the remainder of the system. When openings are required, SSPWs can be subdivided in smaller SSPW segments by using HBEs and VBEs bordering the openings. SSPWs with holes in the web not surrounded by HBEs/VBEs have not been tested. The provisions will allow other openings that can be justified by analysis or testing.

8.7.4.4 Maximum Slenderness Ratio for Plates. A limit on the slenderness of plate elements in SSPW is needed to ensure that adequate cyclic ductility is provided. The limit provided is consistent with successful tests of this system.

8.7.5 Connections of Webs to Boundary Elements. The required strength of web connections to the surrounding HBEs and VBEs shall develop the expected tensile strength of the webs. Net-sections shall also provide this strength for the case of bolted connections.

The strip model can be used to model the behavior of SSPWs and the tensile yielding of the webs at angle α. A single angle of inclination taken as the average for all the panels may be used to analyze the entire wall. The expected tensile strength of the web strips shall be defined as

$$R_y F_y A_s$$

where A_s is the area of a strip, equal to

$$A_s = (L \cos \alpha + H \sin \alpha) / n$$

where L and H are the width and height of a panel, n is the number of strips per panel, and n shall be taken greater than or equal to 10.

This analysis method has been shown, though correlation with physical test data, to adequately predict SSPW performance. It is recognized, however, that other advanced analytical techniques (such as the Finite Element Method (FEM)) may also be used for design of SSPWs. If such non-linear (geometric and material) FEM models are used, they should be calibrated against published test results to ascertain reliability for application.

8.7.6 Horizontal and Vertical Boundary Elements (HBEs and VBEs). Per capacity design principles, all edge boundary elements (HBEs and VBEs) shall be designed to resist the maximum forces generated by the tension field action of the webs fully yielding. Axial forces, shears, and moments develop in the boundary elements of the SSPW as a result of the response of the system to the overall overturning and shear, and this tension field action in the webs. Actual web thickness must be considered for this calculation, because webs thicker than required may have be used due to availability, or minimum thickness required for welding.

At the top panel of the wall, the vertical components of the tension field shall be anchored to the HBE. The HBE shall have sufficient strength to allow development of full tensile yielding across the panel width.

At the bottom panel of the wall, the vertical components of the tension field shall be anchored to the HBE. The HBE shall have sufficient strength to allow development of full tensile yielding across the panel width. This may be accomplished by continuously anchoring the HBE to the foundation.

For intermediate HBEs of the wall, the anticipated variation between the top and bottom web normal stresses acting on the HBE is usually small, or null when webs in the panel above and below the HBE have identical thickness. While top and bottom HBEs are typically of substantial size, intermediate HBEs are relatively smaller.

Beyond the exception mentioned in Sec. 8.7, in some instances, the engineer may be able to justify yielding of the boundary elements by demonstrating that the yielding of this edge boundary element will not cause reduction on the SSPW shear capacity to support the demand and will not cause a failure in vertical gravity carrying capacity.

Forces and moments in the members (and connections), including those resulting from tension field action, may be determined from a plane frame analysis. The web is represented by a series of inclined pin-ended strips, as described in C8.7.4-1. A minimum of 10 equally spaced pin-ended strips per panel will be used in such an analysis.

A number of analytical approaches are possible to achieve capacity design and determine the same forces acting on the vertical boundary elements. Some example methods applicable to SSPWs follow. In all cases, actual web thickness must be considered, for reasons described earlier.

Non-linear push-over analysis

A model of the SSPW can be constructed in which bi-linear elasto-plastic web elements of strength $R_yF_yA_s$ are introduced in the direction α. Bi-linear plastic hinges can also be introduced at the ends of the horizontal boundary elements. Standard push-over analysis conducted with this model will provide axial forces, shears, and moments in the boundary frame when the webs develop yielding. Separate checks are required to verify that plastic hinges do not develop in the horizontal boundary elements, except at their ends.

Combined linear elastic computer programs and capacity design concept

The following four-step procedure provides reasonable estimates of forces in the boundary elements of SSPW systems.

Lateral Forces: Use combined model, boundary elements and web elements, to come up with the demands in the web and the boundary elements based on the code required base shear. The web elements shall not be considered as vertical-load carrying elements.

Gravity Load (Dead Load and Live Load): Apply gravity loads to a model with only gravity frames. The web elements shall not be considered as vertical-load carrying elements.

Without any overstrength factors, design the boundary elements using the demands based on combination forces of the above steps 1 and 2.

Boundary Element Capacity Design Check: Check the boundary element for the maximum capacity of the web elements in combination with the maximum possible axial load due to over-turning moment. Use the axial force obtained from step 1 above and multiply by overstrength factor Ω_o. Apply load from web elements ($R_y F_y A_s$) in the direction of α. For this capacity design check use material strength reduction factor of 1.0.

Indirect Capacity Design Approach

The CSA-S16-02 (CSA 2002) proposes that loads in the vertical boundary members can be determined from the gravity loads combined with the seismic loads increased by the amplification factor,

$$B = V_e / V_u$$

where

V_e = expected shear strength, at the base of the wall, determined for the web thickness supplied

= $0.5\, R_y\, F_y\, t_w\, L \sin 2\alpha$

V_u = factored lateral seismic force at the base of the wall

In determining the loads in VBEs, the amplification factor, B, need not be taken as greater than R.

The VBE design axial forces shall be determined from overturning moments defined as follows:

(i) the moment at the base is $B{\cdot}M_u$, where M_u is the factored seismic overturning moment at the base of the wall corresponding to the force V_u;

(ii) the moment $B{\cdot}M_u$ extends for a height H but not less than two stories from the base; and

(iii) the moment decreases linearly above a height H to B times the overturning moment at one story below the top of the wall, but need not exceed R times the factored seismic overturning moment at the story under consideration corresponding to the force V_u.

The local bending moments in the VBEs due to tension field action in the web shall be multiplied by the amplification factor B.

Preliminary Design

For preliminary proportioning of HBEs, VBEs, and webs, a SSPW wall may be approximated by a vertical truss with tension diagonals. Each web is represented by a single diagonal tension brace within the story. For an assumed angle of inclination of the tension field, the web thickness, t_w, may be taken as:

$$t_w = \frac{2A\Omega_s \sin\theta}{L \sin 2\alpha}$$

where

A = area of the equivalent tension brace

θ = angle between the vertical and the longitudinal axis of the equivalent diagonal brace

L = the distance between VBE centerlines

α = assumed angle of inclination of the tension field measured from the vertical per Sec. 8.7.4

Ω_s = the system overstrength factor as defined by FEMA 369 and taken as 1.2 for SSPWs (Berman and Bruneau 2003).

Determination of A is originally estimated from an equivalent brace size to meet the structure's drift requirements.

8.7.6.3 Boundary Element Compactness. Some amount of local yielding is expected in the HBEs and VBEs to allow development of the plastic mechanism of SSPW systems. For that reason, HBEs and VBEs shall comply with the requirements in AISC Seismic Table I-8-1 for SMFs.

8.7.6.5 Lateral Bracing. Providing stability of SSPW systems boundary elements is necessary for proper performance of the system. The lateral bracing requirements for HBEs are provided to be consistent with beams in SMFs for both strength and stiffness. In addition, all intersections of HBEs and VBEs must be braced to ensure stability of the entire panel.

8.7.6.7 Panel Zone. Panel zone requirements are not imposed for intermediate HBEs. These are expected to be small HBEs connecting to sizeable VBEs. The engineer should use judgment to identify special situations in which the panel zone adequacy of VBEs next to intermediate HBEs should be verified.

8.7.6.8 Stiffness of Vertical Boundary Elements. This requirement is intended to prevent excessive in-plane flexibility and buckling of VBEs. Opportunity exists for future research to confirm or improve the applicability this requirement.

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