Chapter 5 Commentary

STRUCTURAL DESIGN CRITERIA

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

5.2 DESIGN BASIS: Structural design for acceptable seismic resistance includes:

1. The selection of vertical and lateral-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 structures' 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 and 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. (The deflection amplification is specified in Sec. 5.4.6.) Considering the intended structural performance and acceptable deformation levels, Sec. 5.2.8 prescribes the story drift limits for the expected (i.e. 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 (e.g., formation of a first plastic hinge in the structure). A structural steel frame comprised of 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 this significant yield when at least one of the sections of its most highly stressed *component* reaches its strength as set forth in Chapter 9. For other structural materials that do not have their sectional yielding capacities as easily defined, modifiers to working stress values are provided. 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 C5.2-1 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.

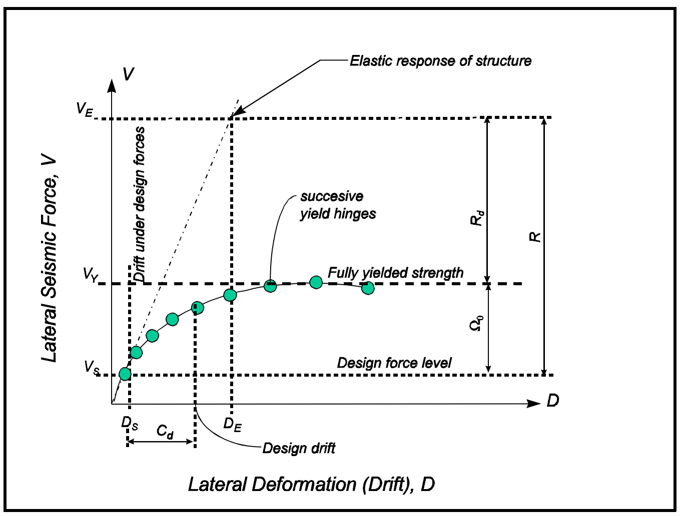

FIGURE C5.2-1 Inelastic force-deformation curve.

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 (i.e. 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, nominally 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 C5.2-1 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 5.2.2, 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 C5.2-1 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 C5.2-1). 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:

$$R = \frac{V_E}{V_S} \qquad \text{(C5.2.1-1)}$$

is always larger then 1.0; thus, all structures are designed for forces smaller than those the design ground motion would produce in a completely linear-elastic responding structure. 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 C.5.2-1) that is significantly lower than the elastic seismic force demand (V_E in Figure C.5.2.1) 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):

$$R_d = \frac{V_E}{V_Y} \qquad \text{(C5.2.1-2)}$$

then it is clear from Figure C5.2-1 that the response modification coefficient, *R*, is the product of the ductility reduction factor and structural overstrength factor (Uang, 1991):

$$R = R_d \Omega_0 \qquad \text{(C5.2.1-3)}$$

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 C5.2-2 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 looses 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.

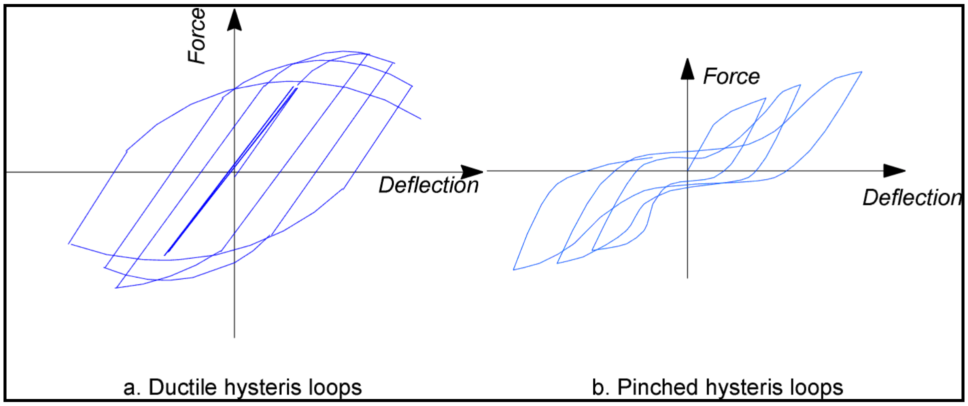

FIGURE C5.2-2 Typical hysteretic curves.

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_o 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. C5.2.1-3). 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 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 5.2.2, to replace the previous 2*R*/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 new coefficient ρ, that is calculated based on percent of the total lateral force resisted by any individual element has been introduced into the *Provisions* in Sec. 5.2.4. 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, that 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 as structures experience greater levels of inelastic behavior, they also experience more 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. 5.2.6 for each Seismic Design Category and the more stringent drift limits in Table 5.2.8. These factors, in addition to strength, are extremely important to obtaining the seismic performance desired for buildings in some Seismic Use Groups.

Sec. 5.2.1 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.

5.2.2 Basic Seismic-Force-Resisting Systems: For purposes of these seismic analyses and design requirements, building framing systems are grouped in the structural system categories shown in Table 5.2.2. 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 5.2.2 continue to be reviewed in light of recent research results. In the selection of the *R* values for the various systems, consideration has been given to the general observed performance of each of the system types during past earthquakes, the general toughness (ability to 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 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 5.2.2 and limited by the Seismic Design Categories.

Special moment frames must meet all the design and detail requirements of Chapter 8, 9, or 10. 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. Note that Sec. 5.2.2 (Table 5.2.2) requires moment frames in Categories D and E or F greater than 160 ft and 100 ft in height, respectively, to be special moment frames.

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 5.2.2 are similar to those for comparable systems of structural steel and reinforced concrete. The values shown in Table 5.2.2 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.

5.2.2.1 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, or 10. The moment frame is required to be capable of resisting at least 25 percent (judgmentally selected) of the specified seismic force. Normally the moment frame would be a part of the basic space frame. The walls or bracing acting together with the moment frame must be capable of resisting all of the design seismic force. The following analyses are required for dual systems:

1. The frame and shear walls or braced frames must resist the prescribed lateral seismic force in accordance with 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.

5.2.2.2 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. 5.2.2.2.1 is to prohibit support of one system by another possessing characteristics that result in a lower base shear factor. The entire system should be designed for the higher seismic shear as the provision stipulates. The exception is included to permit the use of such systems as a braced frame penthouse on a moment frame building in which the mass of the penthouse does not represent a significant portion of the total building and, thus, would not materially affect the overall response to earthquake motions.

Sec. 5.2.2.2.2 pertains to details and is included to help ensure that the more ductile details inherent with the design for the higher *R* value system will be employed throughout. The intent is that details common to both systems be designed to remain functional throughout the response in order to preserve the integrity of the seismic-force-resisting system.

5.2.2.3 - 5.2.2.6 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. 5.2.6 and Chapters 8 through 14. Any type of building framing system permitted by the *Provisions* may be used for Categories A, B, and C except frames limited to Category A or Categories A and B only by the requirements of Chapters 9 and 12. Limitations regarding the use of different structural systems are given for Categories D, E and F.

5.2.2.4 Seismic Design Categories D and E: Sec. 5.2.2.4 covers Categories D and E, which compares roughly to California design practice for normal buildings other than hospitals. According to the requirements of Chapters 8 and 9, all moment-resisting frames of steel or concrete must be special moment frames. Note that present SEAOC and *UBC* recommendations have similar requirements for concrete frames; however, ordinary moment frames of structural steel may be used for heights up to 160 ft (49 m). In keeping with the philosophy of present codes for zones of high seismic risk, these 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.

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

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

2. A dual system as defined in the Glossary, wherein the prescribed forces are resisted by the entire system and the special moment frame is designed to resist at least 25 percent of the prescribed seismic force. This requirement is also similar to SEAOC and *UBC* recommendations. The purpose of the 25 percent frame is to provide a secondary defense system with higher degrees of redundancy and ductility in order to improve the ability of the building to support the service loads (or at least the effect of gravity loads) after strong earthquake shaking. It should be noted that SEAOC and *UBC* 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 (see Table 5.2.2). If the analysis of the interacting behavior is based only on the seismic lateral force vertical distribution recommended in the equivalent lateral force procedure of Sec. 5.3, the interpretation of the results of this analysis for designing the shear walls or braced frame should recognize the effects of higher modes of vibration. The internal forces that can be developed in the shear walls in the upper stories can be more severe than those obtained from such analysis.

3. The use of a shear wall (or braced frame) system of cast-in-place concrete or structural steel up to a height of 240 ft (73 m) is permitted only if braced frames or shear walls in any plane do not resist more than 50 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 C5.2.2.4-1) is acceptable according to the *Provisions*, it is highly recommended that use of such a system be avoided, particularly for taller buildings. The intent is to replace it by the system with lateral force resistance distributed across the entire building (Figure C5.2.2.4-2). The latter system is believed to be more suitable in view of the lack of reliable data regarding the behavior of tall buildings having structural systems based on central cores formed by coupling shear walls or slender braced frames.

5.2.2.4.2 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

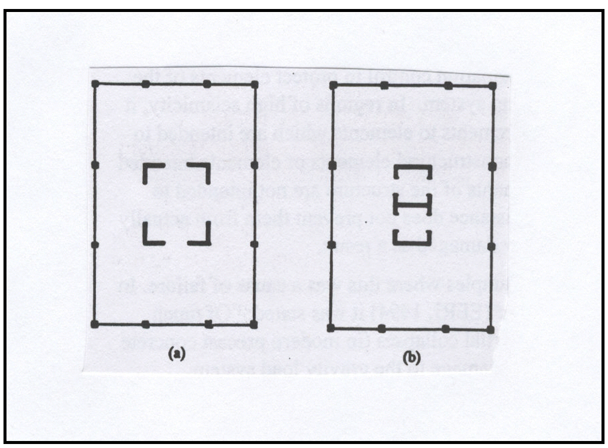

Figure C5.2.2.4-1 Arrangement of shear walls and braced frames – not recommended. Note that the heavy lines indicate shear walls and/or braced frames.

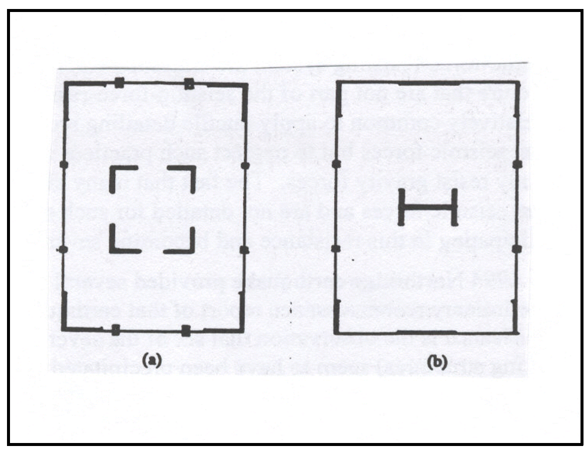

Figure C5.2.2.4-2 Arrangement of shear walls and braced frames – recommended. Note that the heavy lines indicate shear walls and/or braced frames.

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 infill against by reducing the effective flexural height of these columns to the height of the openings in the walls. If these walls are not uniformly distributed throughout the structure, or not 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 seismic 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 torisional irregularities. This section requires consideration of these potential effects.

5.2.2.4.3 Deformational Compatibility: 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 only resist 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 participating in 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 new language in Sec. 5.2.2.4.3 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. 5.2.4.3 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.

5.2.2.5 Seismic Design Category F: Sec. 5.2.2.5 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 protecting 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. 5.2.6 and Chapters 8 through 12.

5.2.3 Structure 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 can not 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.

5.2.3.2 Plan Irregularity: Sec. 5.2.3.2 indicates, by reference to Table 5.2.3.2, when a building must be designated as having a plan irregularity for the purposes of the *Provisions*. A building may have a symmetrical geometric shape without re-entrant corners or wings but still be classified as irregular in plan because of distribution of mass or vertical seismic resisting elements. Torsional effects in earthquakes can occur even when the static centers of mass and resistance coincide. For example, ground motion waves acting with a skew with respect to the building axis can cause torsion. Cracking or yielding in a nonsymmetrical fashion also can cause torsion. These effects also can magnify the torsion due to eccentricity between the static centers. For this reason, buildings having an eccentricity between the static center of mass and the static center of resistance in excess of 10 percent of the building dimension perpendicular to the direction of the seismic force should be classified as irregular. The vertical resisting *components* may be arranged so that the static centers of mass and resistance are within the limitations given above and still be unsymmetrically arranged so that the prescribed torsional forces would be unequally distributed to the various *components*. 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. 5.3.5.2.

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 C5.2.3.2.

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

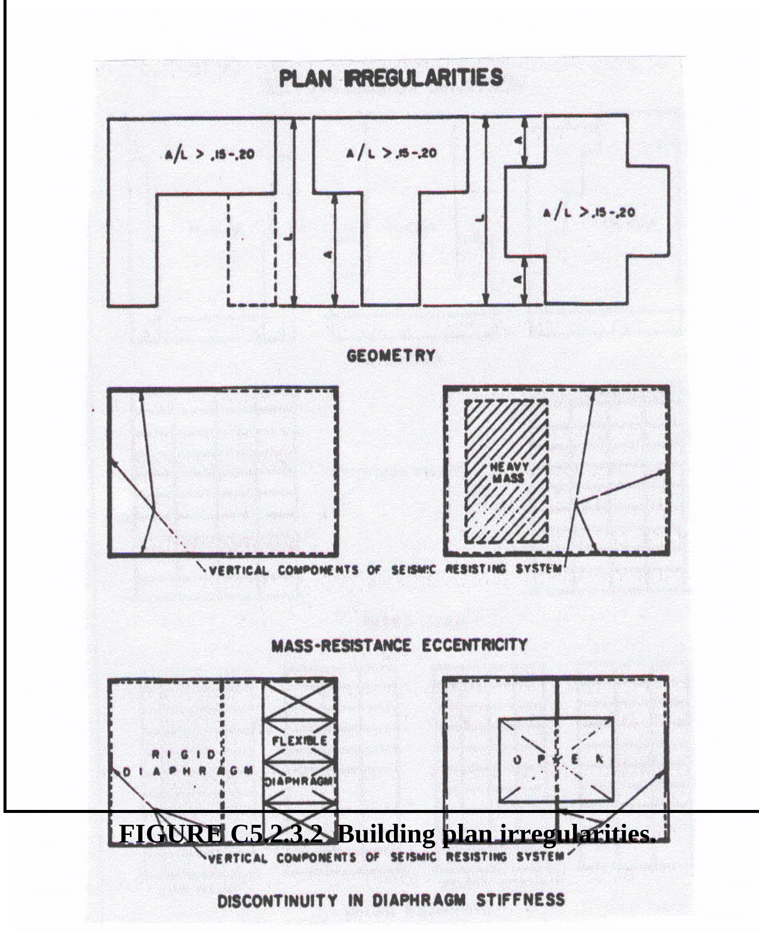

FIGURE C5.2.3.2 Building plan irregularities.

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

5.2.3.3 Vertical Irregularity: Sec. 5.2.3.3 indicates, by reference to Table 5.2.3.3, when a structure must be considered to have a vertical irregularity. Vertical configuration irregularities affect the responses at the various levels and induce loads at these levels that are significantly different from the distribution assumed in the equivalent lateral force procedure given in Sec. 5.3.

A moment resisting frame building might be classified as having a vertical irregularity if one story were much taller than the adjoining stories and the resulting decrease in stiffness that would normally occur was not, or could not be, compensated for. Examples of vertical irregularities are illustrated in Figure C5.2.3.3.

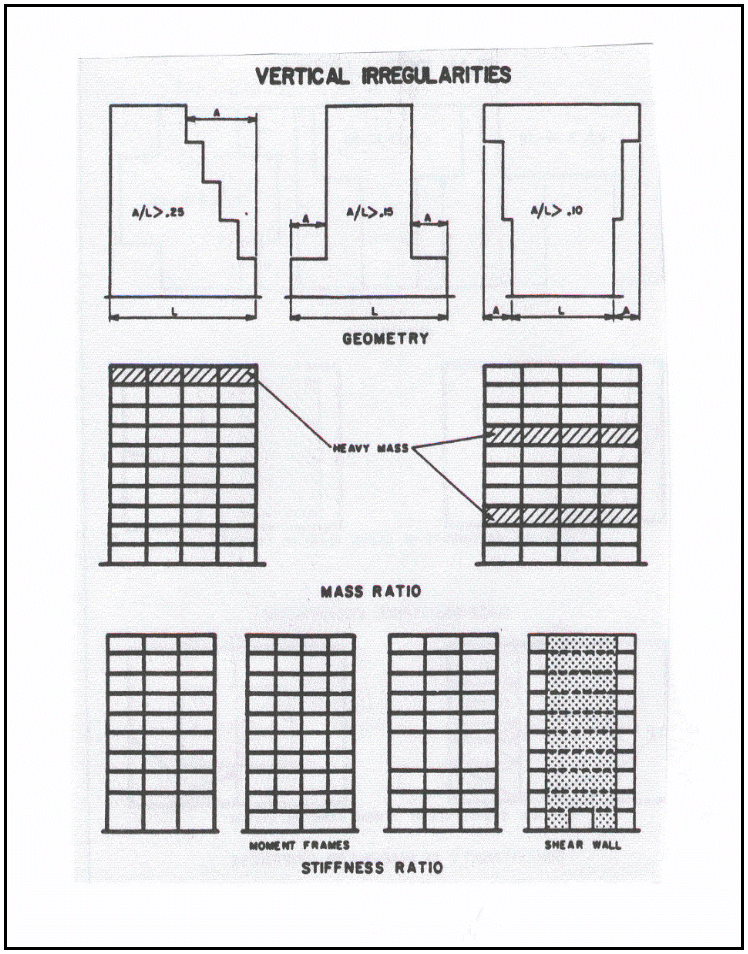

FIGURE C5.2.3.3 Building elevation irregularities.

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. 5.2.6.2.4 when 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.

5.2.4 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. 5.2.1 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. C5.2.1. 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 reliability factor, ρ, that is applied to the design earthquake loads in the basic load combination equations of Sec. 5.2.7, 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, the element demand/capacity ratios, types of mechanisms which may form, the individual characteristics of building systems and materials, building height, number of stories, irregularity, torsional resistance, chord and collector length, diaphragm spans, the number of lines of resistance, and the 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 one-third height. However, many failures of buildings have occurred at and above mid-heights. Therefore, the *Provisions* base the ρ factor on the worst "r" for the least redundant story, which should then be 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.

5.2.5 Structural Analysis: 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. 5.4).
2. Modal analysis procedure (response spectrum analysis) (Sec. 5.5).
3. Linear response history analysis (Sec. 5.6).
4. Inelastic 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 5).
5. Inelastic response history analysis involving step-by-step integration of the coupled equations of motion (Sec. 5.7).

Each procedure becomes more rigorous if effects of soil-structure interaction are considered, either as presented in Sec. 5.8 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*).

The equivalent lateral force (ELF) procedure specified in Sec. 5.4 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, that more closely adopts the direct consideration of 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, it 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. 5.5 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. 5.4 and the modal analysis procedure of Sec. 5.5 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 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. 5.4.3. 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. 5.2.3.2) 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 translational and one 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 translation and three rotational) connected by generalized stiffness elements.

The ELF procedure (Sec. 5.4) and the modal analysis 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 inelastic 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, the motion in the direction along which the structure is being analyzed; otherwise at least three degrees of freedom per floor, two translational motions and one torsional, should be included. It should be recognized that the results of a nonlinear response history analysis of such mathematical 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 member hysteretic behavior.

Because of these sensitivities and limitations, the maximum base shear produced in an inelastic analysis should not be less than that required by Sec. 5.4.

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. 5.2.3.

Neither regular nor irregular buildings in Seismic Design Category A are required to be analyzed as a whole for seismic forces, but certain minimum requirements are given in Sec. 5.2.5.1. In addition, there is a requirement that Seismic Design Category A structure should be evaluated for a total lateral force equal to a nominal percentage of their effective weight. The purpose of this provision is to assure that a complete lateral-force-resisting system is provided for all structures. Although this requirement was first introduced in the 1997 edition of the *Provisions*, in the 2000 edition it was formalized and termed the Index force Procedure (Sec. 5.3).

For the higher Seismic Design Categories, 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 5.2.5.1. 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 it 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. 5.3.4-1 and 5.3.4-2) 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. 5.4. 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. 5.4.
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. 5.3 with the base taken at the top of the base portion.
   b. The base portion then must be analyzed in accordance with the procedures in Sec. 5.3 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. 5.5 include a simplified version of modal analysis that accounts for irregularity in mass and stiffness distribution over the height of the building. It would be adequate, in general, to use the ELF procedure for *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. 5.5 should be used:

1. Compute the story shears using the ELF procedure specified in Sec. 5.4.
2. On this basis, approximately dimension the structural members, and then compute the lateral displacements of the floor.
3. Replace h in Eq. 5.4.3-2 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. 5.4 by more than 30 percent, the building should be analyzed using the procedure of Sec. 5.5. If the difference is less than this value, the building may be designed using

the story shear obtained in the application of the present criterion and the procedures of Sec. 5.5 are not required.

Application of this procedure to these structures requires far less computational effort than the use of the modal analysis procedure of Sec. 5.5. 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.

5.2.5.2. 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 maximum simultaneously. This section provides a reasonable and adequate method for combining them. It requires that structural elements be designed for 100 percent of the effects of seismic forces in one principal direction combined with 30 percent of the effects of seismic forces in the orthogonal direction.

The following combinations of effects of gravity loads, effects of seismic forces in the x-direction, and effects of seismic forces in the y-direction (orthogonal to x-direction) thus pertain:

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

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

5.2.6 Design and Detailing Requirements: The design and detailing requirements for *components* of the seismic-force-resisting system are stated in this section. The combination of load effects is specified in Sec. 5.2.7. 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 4.2.1a and 4.2.1b), which relate to the design ground motion severities, given by the spectral response acceleration coefficients S_{DS} and S_{D1} (Sec. 4.1.1) and the Seismic Use Group (Sec. 1.3).

5.2.6.1 Seismic Design Category A: Because of the very low seismicity associated with sites with S_{DS} less than 0.25g and S_{D1} less than 0.10g , it is considered appropriate for Category A buildings to require only a complete lateral-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 lateral-force-resisting system, ties, and wall anchorage are required by these *Provisions*.

5.2.6.1.1 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 buildings. 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 building is that it is tied together to act as a unit. This attribute not only is important in earthquake-resistant design, but also is indispensable in resisting high winds, floods, explosion, progressive failure, and even such ordinary hazards as foundation settlement. Sec. 5.2.6.1.1 requires that all parts of the building (or unit if there are separation joints) be so tied together that any part of the structure is tied to the rest to resist a force of $S_{DS}/7.5$ (with a minimum of 5 percent g) times the weight of the smaller. In addition, beams must be tied to their supports or columns and columns to footings for a minimum of 5 percent of the dead and live load reaction.

Certain connections of buildings with plan irregularities must be designed for higher forces than calculated due to the simplifying assumptions used in the analysis by Sec. 5.3, 5.4, and 5.5 (see Sec. 5.2.6.4.2).

5.2.6.1.2 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} Newtons per meter (N/m). This requirement alone may not provide complete earthquake-resistant design, but observations of earthquake damage indicate that it can greatly increase the earthquake resistance of buildings and reduce hazards in those localities where earthquakes may occur but are rarely damaging.

5.2.6.2 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.

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

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

Redundancy often is accomplished by making all joints of the vertical load-carrying frame moment resisting and incorporating them into the seismic-force-resisting system. These multiple points of resistance can prevent a catastrophic collapse due to distress or failure of a member or joint. (The overstrength characteristics of this type of frame were discussed in the commentary on Sec. 5.2.1.)

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

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

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

5.2.6.2.5 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 C5.2.6.2.5.

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 C5.2.6.2.5 reinforcing is required to collect the shears or drag the forces from the diaphragm into the shear wall. Similar collector elements are needed in most shear walls and some frames.

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

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

When openings occur in shear walls, diaphragms, etc., it is not adequate to only provide temperature trim bars. 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.

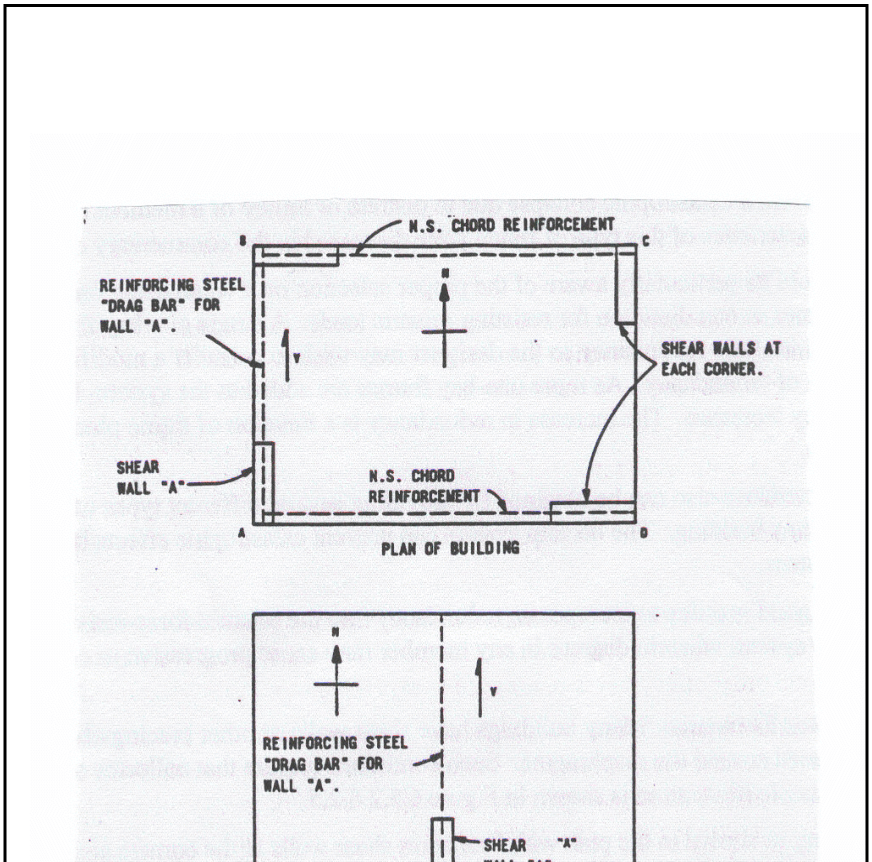

FIGURE C5.2.6.2.5 Collector element used to (a) transfer shears and (b) transfer drag forces from diaphragm to shear wall.

5.2.6.2.7 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.

5.2.6.2.8 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. 5.4.1 and 5.4.4 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. 5.4.1 and 5.4.4. 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. 5.4.1 and 5.4.4 is based on analyses of inverted pendulums covering a wide range of practical conditions.

5.2.6.2.9 Anchorage of Nonstructural Systems: Anchorage of nonstructural systems and *components* of buildings is required when prescribed in Chapter 6.

5.2.6.3 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 inter-connection between pile caps and caissons is required.

5.2.6.4 Seismic Design Category D: 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.

5.2.7 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 dead, earthquake, live, and snow loads.

The design basis expressed in Sec. 5.2.1 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. 5.2.7-1 and 5.2.7-2.

In Eq. 5.2.7-1 and 5.2.7-2 , 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. 5.2.7-1 and 5.2.7-2 in the 1997 *Provisions*. This factor, determined in accordance with Sec. 5.2.4, relates to the redundancy inherent in the lateral-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 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 from 6 to 23 stories in height. 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.

The special load combination of Sec. 5.2.7.1 is intended to address those situations where failure of an isolated, individual, brittle element can result in the loss of a complete lateral-force-resisting system or in instability and collapse. This section has evolved over several editions. In the 1991 Edition, a $2R/5$ factor was introduced to better represent the behavior of elements sensitive to overstrength in the remainder of the seismic resisting system or in specific other structural *components*. The particular number was selected to correlate with the $3R_w/8$ factor that had been introduced in Structural Engineers Association of California (SEAOC) recommendations and the *Uniform Building Code*. 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 5.2.2, dependent on the individual system.

The special load combinations of Eq. 5.2.7.1-1 and 5.2.7.1-2 were first introduced in the 1991 Edition of the *Provisions*, for the design of elements that could fail in an undesirable manner when subjected to demands that are significantly larger than those used to proportion them. It recognizes the fact that the actual response (forces and deformations) developed by a structure subjected to the design earthquake ground motion will be substantially larger than that predicted by the design forces. Through the use of the Ω_o coefficient, this special equation provides an estimate of the maximum forces actually likely to be experienced by an element.

When originally introduced in the 1991 *Provisions*, the overstrength factor Ω_o was represented by the factor $2R/5$. That particular value was selected to correlate with the $3R_w/8$ factor that had been previously introduced in Structural Engineers Association of California (SEAOC) recommendations and the *Uniform Building Code* in 1988. Typically, both of these factors resulted in a three to four fold amplification in the design force levels, based on the historic judgment that the real forces experienced by a structure in a major earthquake are probably on the order of 3 to 4 times the design force levels.

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_o 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 lateral-force-resisting system, will fall within a range of overstrength values. Since the purpose of the Ω_0 factor in Eq. 5.2.7.1-1 and 5.2.7.1-2 is to estimate the maximum force that can be delivered to a component that is

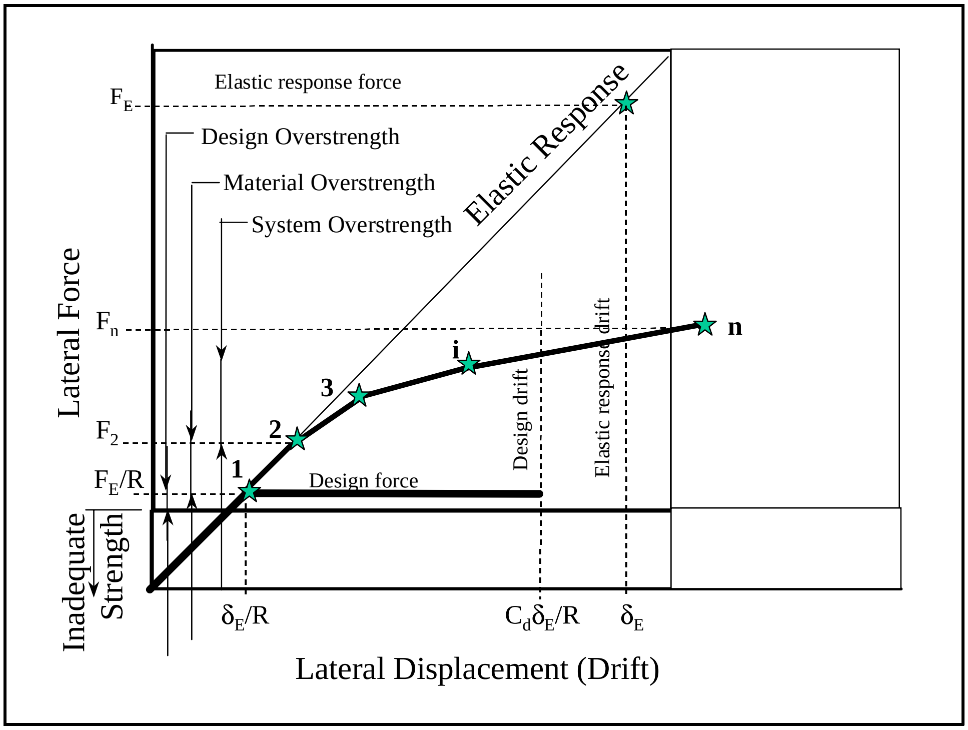

FIGURE C5.2.7 Factors affecting overstrength.

sensitive to overstress, the values of this factor tabulated in Table 5.2.2 are intended to be representative of the larger values in this range for each system.

Figure C5.2.7 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.

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 is also potentially regionally dependent. The SAC study was conducted for frames in Seismic Design Category 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 are 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 seek to intentionally 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 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, since once the brace in the frame yields, the system becomes fully yielded. For structures that have a number of elements participating in lateral seismic 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 concentric braced frames and eccentric 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 affect 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 C5.2.7-1 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 C5.2.7-1 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 & 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 5.2.2, designers are encouraged to base the maximum forces used in Eqs. 5.2.7.1-1and 5.2.7.1-2 on the results of a 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 time 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 time history analyses are utilized, the forces utilized for design as the maximum force, should probably be that 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. 5.2.7.1-1 and 5.2.7.1-2.

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 these special load combinations 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 for conditions where the isolated independent failure of any element can lead to a condition of instability or collapse and should use the special load combinations of Eq. 5.2.7.1-1 and 5.2.7.1-2 for the design of these 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.

5.2.8 Deflection and Drift Limits: This section provides procedures for the limitation of story drift. The term "drift" has two connotations:

1. "Story drift" is the maximum lateral displacement within a story (i.e., the displacement of one floor relative to the floor below caused by the effects of seismic loads).
2. The lateral displacement or deflection due to design forces is the absolute displacement of any point in the structure relative to the base. This is not "story drift" and is not to be used for drift control or stability considerations since it may give a false impression of the effects in critical stories. However, it is important when considering seismic separation requirements.

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

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

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

The design story drift limits of Table 5.2.8. 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 5.2.8. are story drifts and, therefore, are applicable to each story (i.e., they must not be exceeded in any story even though the drift in other stories may be well below the limit.) The limit, Δ_a is to be compared to the design story drift as determined by Sec. 5.4.6.1.

Stress or strength limitations imposed by design level forces occasionally may provide adequate drift control. However, it is expected that the design of moment resisting frames, especially steel building frames, and the design of tall, narrow shear wall or braced frame buildings will be governed at least in part by drift considerations. In areas having 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. 5.3 ELF procedure may be too conservative for drift design of very tall moment-frame buildings. It is suggested for these buildings, where the first mode would be responding in the constant displacement region of a response spectra (where displacements would be essentially independent of stiffness), that the modal analysis procedure of Sec. 5.5 be used for design even when not required by Sec. 5.2.5.

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

Although the *Provisions* do not give precise formulations for the separations, it is required that the distance be "sufficient to avoid damaging contact under total deflection" in order to avoid interference and possible destructive hammering between buildings. It is recommended that the distance be equal to the total of the lateral deflections of the two units assumed deflecting toward each other (this involves increasing separations with height). If the effects of hammering can be shown not to be detrimental, these distances can be reduced. For very rigid shear wall structures with rigid diaphragms whose lateral deflections cannot be reasonably estimated, it is suggested that older code requirements for structural separations of at least 1 in. (25 mm) plus ½ in. (13 mm) for each 10 ft (3 m) of height above 20 ft (6 m) be followed.

5.3 INDEX FORCE ANALYSIS PROCEDURE: 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 index force analysis procedure 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 gravity load W is the total weight of the building and that part of the service load that might reasonably be expected to be attached to the building at the time of an earthquake. It includes permanent and movable partitions and permanent equipment such as mechanical and electrical equipment, piping, and ceilings. The normal human live load is taken to be negligibly small in its contribution to the seismic lateral forces. Buildings designed for storage or warehouse usage should have at least 25 percent of the design floor live load included in the weight, W. Snow loads up to 30 psf (1400 Pa) are not considered. Freshly fallen snow would have little effect on the lateral force in an earthquake; however, ice loading would be more or less firmly attached to the roof of the building and would contribute significantly to the inertia force. For this reason, the effective snow load is taken as the full snow load for those regions where the snow load exceeds 30 psf with the proviso that the local 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.

5.4 EQUIVALENT LATERAL FORCE PROCEDURE: This section discusses the equivalent lateral force (ELF) procedure for seismic analysis of structures.

5.4.1 Seismic Base Shear: The heart of the ELF procedure is Eq. 5.4.1.-1 for base shear, which gives the total seismic design force, V, in terms of two factors: a seismic response coefficient, C_s, and the total gravity load of the building, W. The seismic response coefficient C_s, is obtained from Eq. 5.4.1.1-1 and 5.4.1.1-2 based on the design spectral response accelerations, S_{DS} and S_{D1}. These acceleration parameters and the derivation of the response spectrum is discussed more fully in the *Commentary* for Chapter 4.

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

Elastic Acceleration Response Spectra: See the *Commentary* for Chapter 4 for a full discussion of the shape of the spectra accounting for dynamic response amplification and the effect of site response.

Elastic Design Spectra: The elastic acceleration response spectra for earthquake motions has a descending branch for longer values of T, the period of vibration of the system, that varies roughly as $1/T$. 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 maintained 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 pervious 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, 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. 5.4.1.1-1) to one of constant response velocity (eq. 5.4.1.1-2).

Equation 5.4.1.1-2 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 more accurately simulate results of an elastic response spectrum analysis, 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 SDF 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 straight forward 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*, it was elected to limit the application of this technique in Seismic Design Categories D, E, and F to those structures 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 technique 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 the dynamic analysis methods 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 also 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 less than the indicated amount.

Response Modification Factor: The factor R in the denominator of Eq. 5.4.1.1-1 and 5.4.1.1-2 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 (i.e., no reduction from the linear elastic response would be allowed). At the other extreme, a heavily damped building structure with a very ductile structural system would be able to withstand deformations considerably in excess of initial yield and would, therefore, justify the assignment of a larger response reduction factor R. Table 5.2.2 in the *Provisions* stipulates R coefficients for different types of building systems using several different structural materials. The coefficient R ranges in value from a minimum of 1-1/4 for an unreinforced masonry bearing wall system to a maximum of 8 for a special moment frame system. The basis for the R factor values specified in Table 5.2.2 is explained in the Sec. 5.2.1.

The effective value of R used in the base shear equation is adjusted by the occupancy importance factor I. The I value, 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. 5.2.8.) and by providing special design and detailing requirements (Sec. 5.2.6) and materials limitations (Chapters 8 through 12).

5.4.2 Period Determination: In the denominator of Eq. 5.4.1.1-2, T is the fundamental period of vibration of the building. It is preferable that this be determined using modal analysis methods and the principals of structural mechanics. However, methods of structural mechanics cannot be employed to calculate the vibration period before a building has been designed. Consequently, this section provides an approximate method that can be used to estimate building period, with minimal information available on the building design. It is based on the use of simple formulas that involve only a general description of the building type (e.g., steel moment frame, concrete moment frame, shear wall system, braced frame) and overall dimensions (e.g., height and plan length) to estimate the vibration period 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 a 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 5.4.2.1-1, 5.4.2.1-2, and 5.4.2.1-3 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 accommodates 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. For buildings whose design are 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 $1/T$ and assuming that the lateral forces are distributed linearly over the height and the deflections are controlled by drift limitations, a simple analysis of the vibration period by Rayleigh's method 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. 5.3.3.1-1 appeared in the *Provisions* in the form:

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

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 of a significant number of these buildings to strong ground shaking to be recorded and the fundamental period of vibration of the buildings to be calculated. Figures C5.4.2.1-1, C5.4.2.1-2, and C5.4.2.1-3, respectively, show plots of these data as a function of building height for three classes of structures. Figure C.5.4.2.1-1 shows the data for moment-resisting concrete frame buildings; Figure C.5.4.2.1-2, for moment-resisting steel frame buildings; and Figure C.5.4.2.1-3, 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, 5.4.2.1-1 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_t and the superscript x given in Table 5.4.2.1 for these moment-resisting frame structures represent the lower bound (mean -1s) fits to the data shown in Figures C5.4.2.1-1 and C.5.4.2.1-2, respectively, for steel and concrete moment frames. Although updated data were available for concrete shear wall strucures, these data do not fit well with an equation of the form of Eq. 5.4.2.1-1. 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. 5.4.2.1-3, 5.4.2.1-4, and 5.4.2.1-5 provide a reasonably good fit to the data. However, the form of these equations is somewhat complex. Therefore, the simpler form of Eq. 5.4.2.1 contained in earlier editions of the *Provisions* was retained with the newer, more accurate formulation presented as an alternative formulation.

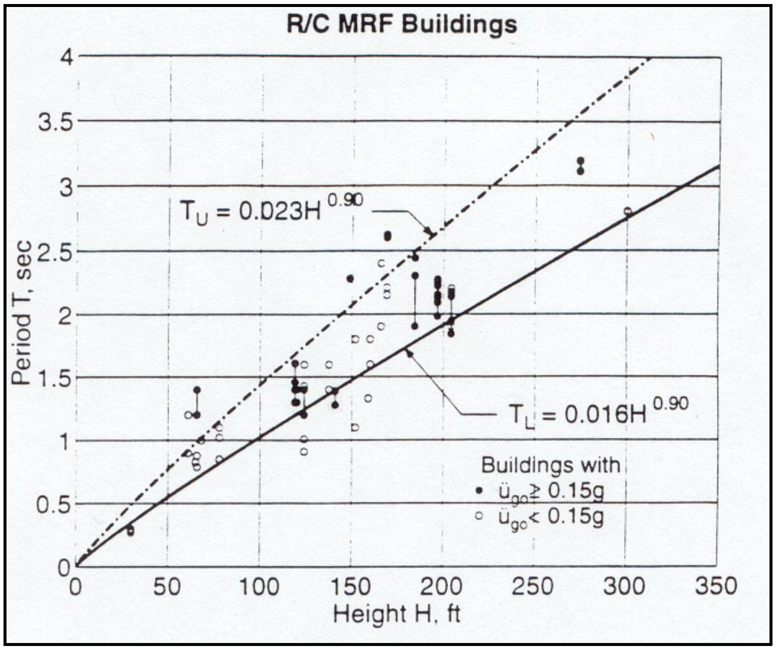

Figure C5.4.2.1-1 Measured building period for reinforced concrete frame structures.

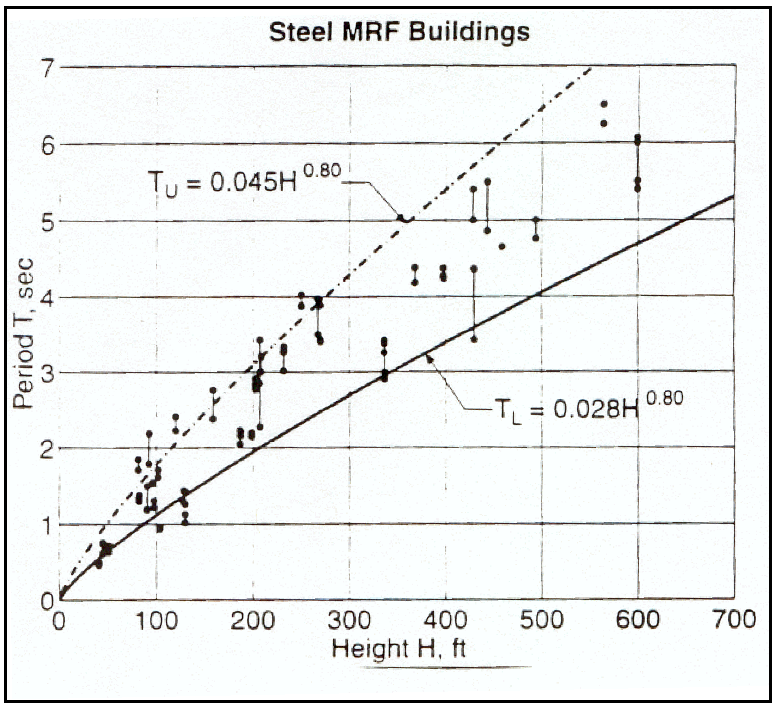

Figure C5.4.2.1-2 Measured building period for moment-resisting steel frame structures.

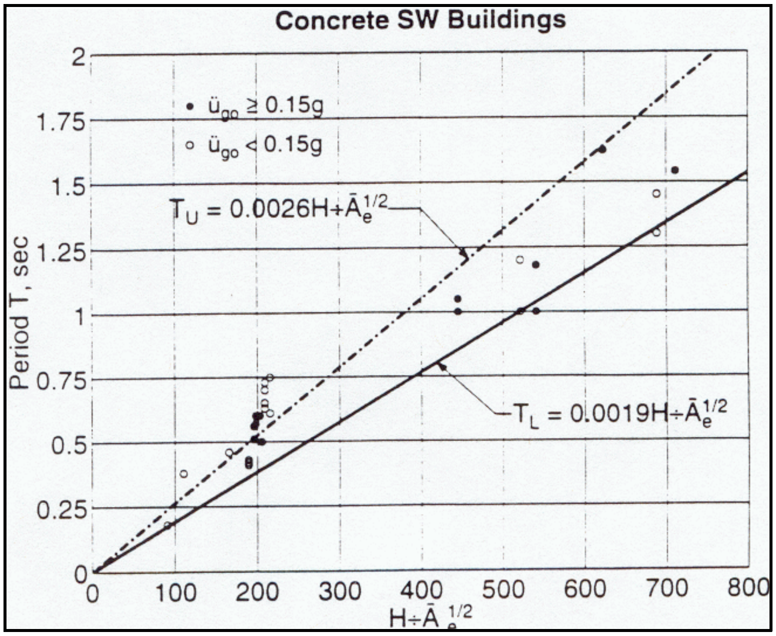

Figure C5.4.2.1-3 Measured building period for concrete shear wall structures.

Updated data for other classes of construction were not available. As a result, the C_t and x values for other types of construction shown in Table 5.4.2.1 are values largely based on limited data obtained from the 1971 San Fernando earthquake that have traditionally been used in the *Provisions*. The optional use of $T = 0.1N$ (Eq. 5.4.2.1-2) is an approximation for low to moderate height frames that has been long in use.

As an exception to Eq. 5.4.2.1-1, these requirements allow the calculated fundamental period of vibration, T, of the seismic-force-resisting system to be used in calculating the base shear. However, the period, T, used may not exceed C_uT_a with T_a determined from Eq. 5.4.2.1-1.

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 C5.4.2.1-1, C5.4.2.1-2, and C5.4.2.1-3 permit direct evaluation of the upper bound on period as a function of the lower bound, given by Eq. 5.4.2.1-1. 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 5.4.2 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. 5.4.2.1-1. For such buildings, it is recommended that the period value T be used in lieu of T_a for calculating the seismic response coefficient, C_s.

Although the approximate methods of Sec. 3.3.3. 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:

$$T = 2\pi \sqrt{\frac{\sum_{i=1}^{n} w_i \delta_i^2}{g \sum_{i=1}^{n} F_i \delta_i}} \qquad \text{(C5.4.2)}$$

where:

F_i = the seismic lateral force at Level *i*,

w_i = the gravity load assigned in Level *i*,

d_i = the static lateral displacement at Level *i* due to the forces F_i computed on a linear elastic basis, and

g = is the acceleration of gravity.

The calculated period increases with an increase in flexibility of the structure because the d 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 d, the deflections are exaggerated and the calculated period is lengthened, leading to a decrease in the seismic response coefficient C_s and, therefore, a decrease in the design force. Nonstructural elements do not know that they are nonstructural. They participate in the behavior of the structure even though the designer may not rely on them for contributing any strength or stiffness to the structure. To ignore them in calculating the period is to err on the unconservative side. The limitation of $C_u T_a$ is imposed as a safeguard.

5.4.3 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 mass and stiffness over the height (see Sec. 5.2.3). 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. 5.4.3-2 is much improved by the procedure described in the last portion of Sec. 5.2.4 of this commentary. The lateral force at each level, *x*, due to response in the first (fundamental) natural mode of vibration is:

$$f_{x1} = V_1 \left(\frac{w_x \phi_{x1}}{\sum_{i=1}^{n} w_i \phi_{i1}} \right) \qquad \text{(C5.4.3)}$$

where:

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

w_i = the weight lumped at the *i*th level, and

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

This is the same as Eq. 5.5.5-2 in Sec. 5.5 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. 5.4.3-2 with $k = 1$ if the first mode shape is a straight line and with $k = 2$ if the first mode shape is a parabola with its vertex at the base.

It is well known that the influence of modes of vibration higher than the fundamental mode is small in the earthquake response of short period 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. 5.3.4-2 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. 5.3.4-2 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.

5.4.4 Horizontal Shear Distribution: The story shear in any story is the sum of the lateral forces acting at all levels above that story. Story x is the story immediately below Level x (Figure C5.4.4). 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.

5.4.4.1 and 5.4.4.2 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, is to be computed as the story shear times the eccentricity perpendicular to the direction of applied earthquake forces.

2. M_{ta}, commonly referred to as "accidental torsion," is to be computed as the story shear times the "accidental eccentricity," equal to 5 percent of the dimension of the 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. 5.4.4 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 (e.g., first, all of them to the left and, then, to the right).

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

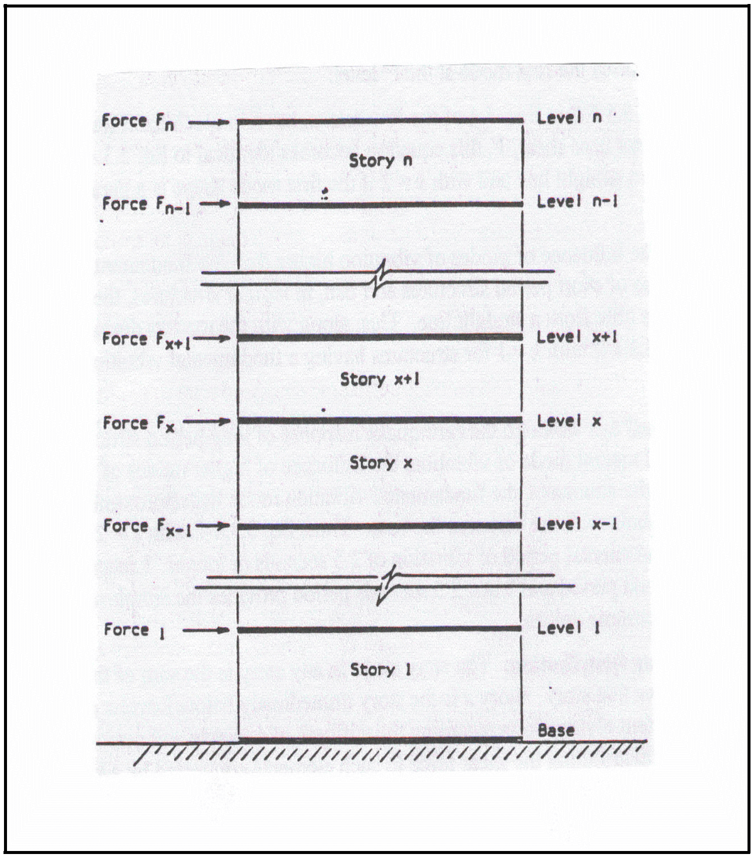

FIGURE C5.4.4 Description of story and level. The shear at Story *x* (*Vx*) is the sum of all the lateral forces at and above Story *x* (F_x through F_n).

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 not smaller than the following two values (Newmark and Rosenblueth, 1971):

1. The story shear times one-half of the maximum of the computed eccentricities in all stories below the one being analyzed and
2. One-half of the maximum of the computed torsional moments for all stories above.

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

There are indications that the 5 percent accidental eccentricity may be too small in some 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 (e.g., those that depend largely on masonry walls for lateral force resistance or those that depend on vertical elements made of different materials), and nominally symmetrical structures that utilize core elements alone for seismic resistance or that behave essentially like elastic nonlinear systems (e.g., some prestressed concrete frames). The amplification factor for torsionally irregular structures (Eq. 5.4.4.1.3-1) was introduced in the 1988 Edition as an attempt to account for some of these problems in a controlled and rational way.

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

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

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

There are some common situations where it is obvious that the diaphragm can be assumed to be either rigid or very flexible in its own plane for purposes of distributing story shear and considering torsional moments. For example, a solid monolithic reinforced concrete slab, square or nearly square in plan, in a 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 should be the envelope of the two sets of forces resulting from both extreme assumptions regarding the diaphragms--rigid or very flexible.

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

5.4.5 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 overturing 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 technique 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 no longer necessary to include this τ coefficient for these structures permitting a significant simplification in the design procedures. 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 the 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. 5.4.1-1. 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.

5.4.6 Drift Determination and *P*-delta Effects: This section defines the design story drift as the difference of the deflections, δ_x, at the top and bottom of the story under consideration. The deflections, δ_x, are determined by multiplying the deflections, δ_{xe} (determined from an elastic analysis), by the deflection amplification factor, C_d, given in Table 5.2.2. 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. 5.4.3. 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. 5.2.7, the deflections, δ_x, may be calculated as indicated above for the seismic-force-resisting system and design forces corresponding to the fundamental period of the structure, T (calculated without the limit $T \le C_u T_a$ specified in Sec. 5.4.2), 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. 5.4.6.2 are significant, the design story drift must be increased by the resulting incremental factor.

The *P*-delta effects in a given story are due to the eccentricity of the gravity load above that story. If the story drift due to the lateral forces prescribed in Sec. 5.4.3 were Δ, the bending moments in the story would be augmented by an amount equal to Δ times the gravity load above the story. The ratio of the *P*-delta moment to the lateral force story moment is designated as a stability coefficient, θ, in Eq. 5.4.6.2-1. If the stability coefficient θ is less than 0.10 for every story, the *P*-delta effects on story shears and moments and member forces may be ignored. If, however, the stability coefficient θ exceeds 0.10 for any story, the *P*-delta effects on story drifts, shears, member forces, etc., for the whole 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 inelastic stiffness in determining *theoretical P*-delta response is unconservative. Given a study carried out by Bernal (1987), it was argued that *P*-delta amplifiers should be based on secant stiffness and that, in other words, the C_d term in Eq.5.4.6.2-1 should be deleted. However, since Bernal's study was based on the inelastic response of single-degree-of-freedom elastic-perfectly plastic systems, significant uncertainties existed regarding the extrapolation of the concepts to the complex hysteretic behavior of multi-degree-of-freedom systems.

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

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

1. Many structures display strength well above the strength implied by code-level design forces (see Figure C5.5.1-1). This overstrength likely protects structures from stability-related failures.2.The likelihood of a stability failure decreases with increased intensity of expected ground-shaking. This is due to the fact that the stiffness of most 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, the requirements of the 1988 Edition of the *Provisions* regarding *P*-delta amplifiers remain in the 1991 and 1994 Editions with the exception that the 0.90 factor in the numerator of the amplifier has been deleted. This factor originally was used to create a transition from cases where *P*-delta effects need not be considered ($\theta \leq 0.10$, amplifier = 1.0) to cases where such effects need be considered ($\theta > 1.0$, amplifier > 1.0).

However, the 1991 Edition introduced a requirement that the computed stability coefficient, θ, not exceed 0.25 or $0.5/\beta C_d$, where βC_d is an adjusted ductility demand that takes into account the fact that the seismic strength demand may be somewhat less than the code strength supplied. The adjusted ductility demand is not intended to incorporate overstrength beyond that computed by the means available in Chapters 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*, $\beta = 1.0$ is obviously conservative.

The principal reason for inclusion of β is to allow for a more equitable analysis of those structures in which substantial extra strength is provided, whether as a result of added stiffness for drift control, from code-required wind resistance, or simply a feature of other aspects of the design. β = story shear demand/story shear capacity is conservatively 1.0 for any design that meets the remainder of the *Provisions*. Some structures inherently possess more strength than required, but instability is not typically a concern for such structures. For many flexible structures, the proportions of the structural members are controlled by the drift requirements rather than the strength requirements; consequently, β is less than 1.0 because the members provided are larger and stronger than required. This has the effect of reducing the inelastic *component* of total seismic drift and, thus, β is placed as a factor on C_d.

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

5.5 MODAL RESPONSE SPECTRUM ANALYSIS PROCEDURE:

5.5.1 General: 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 Equivalent Lateral Force procedure of Sec. 5.4 is simply a first mode application of this technique, that 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. 5.4. Thus, the coefficient C_{sm} in Eq. 5.5.4-1 and the distribution equations, Eq. 5.5.5-1 and 5.5.5-2, are the counterparts of Eq. 5.4.3-1 and 5.4.3-2. This correspondence helps clarify the fact that the simplified modal analysis contained in Sec. 5.5 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. 5.4.

5.5.2 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.

5.5.3 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} from Eqs. 5.5.4. 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.

5.5.4 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, interstory drifts, floor displacements, base (overturning) moments, etc. In this section, the base shear in the m^{th} mode is specified as the product of the modal seismic coefficient C_{sm} and the effective weight W_m for the mode. The coefficient C_{sm} is determined for each mode from Eq. 5.5.4-3 using the associated period of the mode, T_m, in addition to the factors C_v and R, which are discussed elsewhere in the *Commentary*. An exception to this procedure occurs for higher modes of those *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. 5.5.4-4 is used. Equation 5.5.4-4 gives values ranging from $S_{DS}/2.5R$ 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 soils with Soil Profile Type D as specified following Eq. 5.5.4 -3, it is seen that the use of Eq. 5.5.4-4, when applicable, reduces the modal base shear. This is an approximation introduced in consideration of the conservatism embodied in using the spectral shape specified by Eq. 5.5.4-3 and its limiting values. The spectral shape so defined is a conservative approximation to average spectra that are known to first ascend, level off, and then decay as period increases. Equation 5.5.4-3 and its limiting values conservatively replace the ascending portion for small periods by a level portion. For soils with Soil Profile Type A, B and C, the ascending portion of the spectra is completed by the time the period reaches a small value near 0.1 or 0.2 second. On the other hand, for soft soils the ascent may not be completed until a larger period is reached. Equation 5.5.4-4 is then a replacement for the spectral shape for soils with Soil Profile Type D, E and F and short periods that is more consistent with spectra for measured accelerations. It was introduced because it was judged unnecessarily conservative to use Eq. 5.5.4-3 for modal analysis in the case of soils with Soil Profile Types D, E, and F. The effective modal gravity load given in Eq. 5.5.4-2 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. 5.4.5-2 gives values of W_m that are independent of how the modes are normalized.

The final equation of this section, Eq. 5.5.4-5, is to be used if a modal period exceeds 4 seconds. It can be seen that Eq. 5.5.4-5 and 5.5.4-3 coincide at $T_m = 4$ seconds so that the effect of using Eq. 5.5.4-5 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.

5.5.5 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. 5.5.5-1 and 5.5.5-2 and are expressed in terms of the gravity load assigned to the floor, the mode shape, and the modal base shear V_m. In applying the forces F_{xm} to the 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. 5.5.5-1 is somewhat different from that usually employed in standard references and shows clearly the relation between the modal forces and the modal base shear. It therefore is a convenient form for calculation and highlights the similarity to Eq. 5.4.3-1 in the ELF procedure.

The modal deflections at each level are specified by Eq. 5.5.5-3. These are the displacements caused by the modal forces F_{xm} considered as static forces and are representative of the maximum amplitudes of modal response for the essentially elastic motions envisioned within the concept of the seismic response modification coefficient R. This is also a logical point to calculate the modal drifts, which are required in Sec. 5.5.7. If the mode under consideration dominates the earthquake response, the modal deflection under the strongest motion contemplated by the *Provisions* can be estimated by multiplying by the deflection amplification factor C_d. It should be noted also that δ_{xm} is proportional to ϕ_{xm} (this can be shown with algebraic substitution for F_{xm} in Eq. 5.5.5-4) and will therefore change direction up and down the structure for the higher modes.

5.5.6 Modal Story Shears and Moments: This section merely specifies that the forces of Eq. 5.5.5-1 should be used to calculate the shears and moments for each mode under consideration. In essence, the forces from Eq. 5.5.5-1 are applied to each mass, and linear static methods are used to calculate story shears and story overturning moments. The base shear that results from the calculation should check with Eq. 5.5.4-1.

5.5.7 Design Values: This section specifies the manner in which the values of story shear, moment, and drift quantities and the deflection at each level are to be combined. The method used, in which the design value is the square root of the sum of the squares of the modal quantities, was selected for its simplicity and its wide familiarity (Clough and Penzien, 1975; Newmark and Rosenblueth, 1971; Wiegel, 1970). In general, it gives satisfactory results, but it is not always a conservative predictor of the earthquake response inasmuch as more adverse combinations of modal quantities than are given by this method of combination can occur. The most common instance where combination by use of the square root of the sum of the squares is unconservative occurs when two modes have very nearly the same natural period. In this case, the responses are highly correlated and the designer should consider combining the modal quantities more conservatively (Newmark and Rosenblueth, 1971). In the 1991 Edition of the *Provisions* the option of combining these quantities by the complete quadratic combination (CQC) technique was introduced. This method provides somewhat better results than the square root of the sum of 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 justified because the modal analysis gives a somewhat more accurate representation of the earthquake response. Some limit to any such possible reduction that may occur from the calculation of longer natural periods is necessary because the actual periods of vibration may not be as long, even at moderately large amplitudes of motion, due to the stiffening effects of elements not a part of the seismic resisting system and of nonstructural and architectural *components*. The limit is imposed by comparison to 85 percent of base shear value computed with the ELF procedure. Where modal analysis predicts response quantities with a total base shear less than 85 percent of that which could be 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 are not permitted.

5.5.8 Horizontal Shear Distribution: This section requires that the design story shears calculated in Sec. 5.5.7 and the torsional moments prescribed in Sec. 5.4.4 be distributed to the vertical elements of the seismic resisting system as specified in Sec. 5.4.4 and as elaborated on in the corresponding section of this commentary.

5.5.9 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. 5.4 (see Sec. 5.4.5 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. 5.4.5 of this commentary.

5.5.10 *P*-Delta Effects: Sec. 5.4.6 of this commentary applies to this section. In addition, to obtain the story drifts when using the modal analysis procedure of Sec. 5.5, the story drift for each mode should be independently determined in each story (Sec. 5.5.5). The story drift should not be determined from the differential combined lateral structural deflections since this latter procedure will tend to mask the higher mode effects in longer period structures.

5.6 LINEAR RESPONSE HISTORY ANALYSIS PROCEDURE: Linear response history analysis, also commonly known as time history analysis, is a numerically complex 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 displacement of each node in the model and the force in each element of model to be calculated 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 other portions. Response history analyses are highly dependent on the characteristics of the individual ground shaking record 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 the difference in response, resulting from differences in record characteristics, to be observed. 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 less than 7 records, the maximum values of the predicted response parameters be used as the design values. When 7 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.

Whenever possible, ground motion records should be scaled form actual recorded earthquake ground motions, obtained from events of similar magnitude to that which controls the design earthquake for the site, and with the instruments being located on sites with similar characteristics and fault distances to that of the building 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 to modal response spectrum analysis is seldom justified and as a result, this procedure is rarely used in the design process. One exception is for the design of *structures* with energy dissipation systems comprised of linear viscous dampers. Linear response history analysis can be used to predict the response of *structures* with such systems, while modal response spectrum analysis can not.

5.7 NONLINEAR RESPONSE HISTORY ANALYSIS: This method of analysis is very similar to linear response history analysis, described in Sec. 5.6 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 effect of element yielding, buckling and other nonlinear behavior on structural response to be directly accounted for in the analysis. It also permits such nonlinear behaviors as foundation rocking, opening and closing of gaps, nonlinear viscous and hysteric damping to be evaluated. 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. 5.6, or even in element hysteric 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 should be considered, as described in Sec. 5.6. It may also be appropriate to perform sensitivity studies, in which the assumed hysteric properties of elements are allowed to vary, within expected bounds, to allow the effects of such uncertainties on predicted response to be evaluated.

Application of nonlinear response history analysis to even the simplest structures requires large, high speed computers and complex computer software that has specifically been 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 off ices can routinely expect to utilize this technique in the design of complex structures.

5.7.3.1 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.5.7.4. 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 special load combinations of Sec. 5.2.7.1. 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 directly used in the evaluation of the elements for adequacy under Sec. 5.2.7.1.

5.7.4 Design Review: The provisions for design using linear methods of analysis including the equivalent lateral force technique of Sec. 5.4 and the modal response spectrum analysis technique of

Sec. 5.5, are highly prescriptive. They limit the modeling assumptions that can be employed as well as the minimum strength and stiffness the structure must posses. Further, the methods used in linear analysis have become standardized in practice such that there is unlikely to be substantial difference between the results obtained from different designers using the same technique to analyze the same structure. 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 judgement 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 judgement applied by the designer when using these methods is appropriate and compatible with those that would be made by other competent practitioners.

5.8 SOIL-STRUCTURE INTERACTION EFFECTS:

5.8.1 General: *Statement of the Problem:* Fundamental to the design requirements presented in Sec. 5.4 and 5.5 is the assumption that the motion experienced by the base of a structure during an earthquake is the same as the "free-field" ground motion, a term that refers to the motion that would occur at the level of the foundation if no structure was present. 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. 5.8 represent the difference in the flexible-base and rigidly supported 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. 5.8 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. 5.4 and 5.5 and are accounted for only implicitly in Sec. 5.8.

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 aproach, 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. 5.8.

Characteristics of Interaction: The interaction effects in the approach used here are expressed by an increase in the fundamental natural period of the structure and a change (usually an increase) in its effective damping. The increase in period results from the flexibility of the foundation soil whereas the change in damping results mainly from the effects of energy dissipation in the soil due to radiation and material damping. These statements can be clarified by comparing the responses of rigidly and elastically supported systems subjected to a harmonic excitation of the base. Consider a linear structure of weight W, lateral stiffness k, and coefficient of viscous damping c (shown in Figure C5.8.1-1) and assume that it is supported by a foundation of weight W_o at the surface of a homogeneous, elastic halfspace.

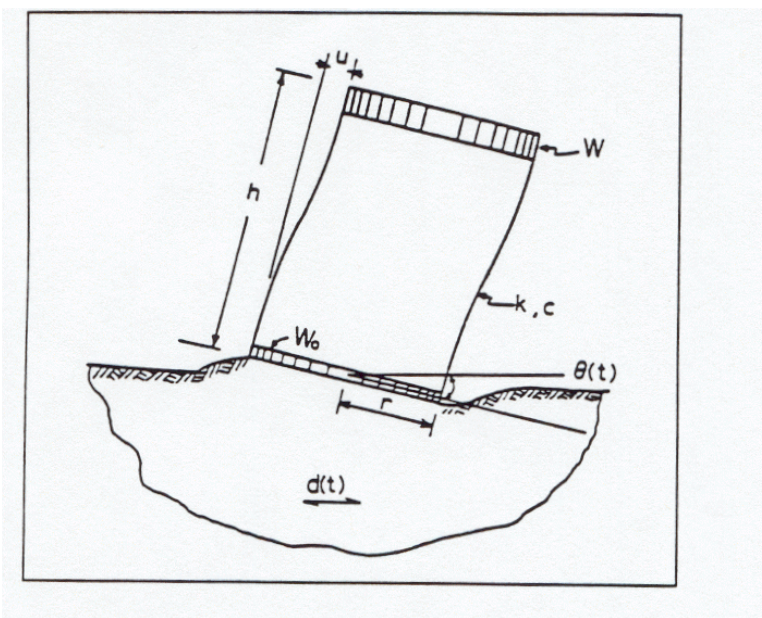

Figure C5.8.1-1 Simple system investigated.

The foundation mat is idealized as a rigid circular plate of negligible thickess bonded to the supporting medium, and the columns of the structure are considered to be weightless and axially inextensible. Both the foundation weight and the weight of the structure are assumed to be uniformly distributed over circular areas of radius r. The base excitation is specified by the free-field motion of the ground surface. This is taken as a horizontally directed, simple harmonic motion with a period T_o and an acceleration amplitude a_m.

The configuration of this system, which has three degrees of freedom when flexibly supported and a single degree of freedom when fixed at the base, is specified by the lateral displacement and rotation of the foundation, y and θ, and by the displacement relative to the base of the top of the structure, u. The system may be viewed either as the direct model of a one-story 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 C5.8.1-2 and C5.8.1-3 represent response spectra for the steady-state amplitude of the total shear in the columns of the system considered in Figure C5.8.1-1. Two different values of *h*/*r* and several different values of the relative flexibility parameter for the soil and the structure, ϕ_o, are considered. The latter parameter is defined by the equation $\delta_o = \frac{h}{v_s T}$ 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 C5.8.1-2 and C5.8.1-3 are displayed in a dimensionless form, with the abscissa representing the ratio of the period of the excitation, T_o, to the fixed-base natural period of the system, *T*, and the ordinate representing the ratio of the amplitude of the actual base shear, *V*, to the amplitude of the base shear induced in an infinitely stiff, rigidly supported structure. The latter quantity is given by the product ma_m, in which $m = W/g$, *g* is the acceleration of gravity, and a_m is the acceleration amplitude of the free-field ground motion. The inclined scales on the left represent the deformation amplitude of the superstructure, *u*, normalized with respect to the displacement amplitude of the free-field ground motion $d_m = \frac{a_m T_0^2}{4\pi^2}$.

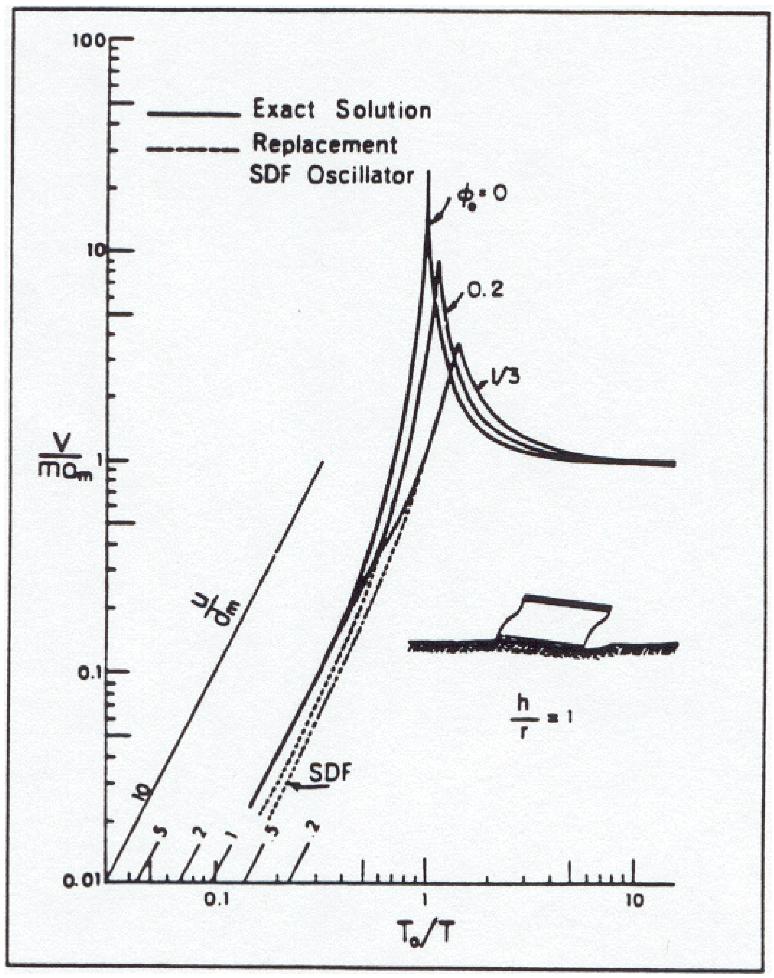

Figure C5.8.1-2 Response spectra for systems with *h/r* = 1 (Veletsos and Meek, 1974).

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_o), can conveniently be expressed by an increase in the natural period of the system over its fixed-base value and by a change in its damping factor.

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

In the context of Fourier analysis, an earthquake motion may be viewed as the result of superposition of harmonic motions of different periods and amplitudes. Inasmuch as the *components* of the excitation with periods close to the resonant period are likely to be the dominant contributors to the response, the maximum responses of the actual system and of the replacement oscillator can be expected to be in satisfactory agreement for earthquake ground motions as well. This expectation has been confirmed by the results of comprehensive comparative studies (Veletsos, 1977; Veletsos and Meek, 1974; Veletsos and Nair, 1975; Jennings and Bielak, 1973).

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

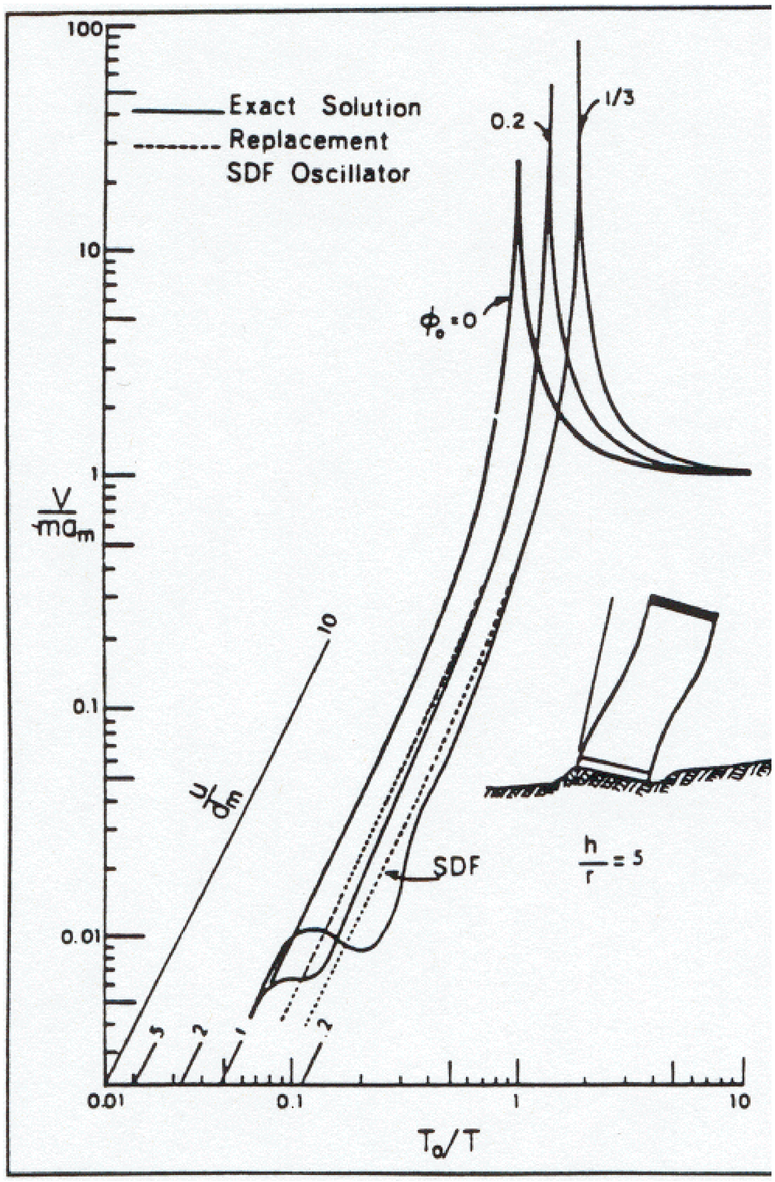

Figure 5.8.1-3 Response spectra for systems with *h/r* = 5 (Veletsos and Meek, 1974).

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. 5.8.2.1.1 and 5.8.2.1.2.

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. 5.8 for the latter cases represent the best interpretration 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 (e.g., to avoid soil instability and settlement associated with the compaction and liquefaction of loose granular soils), is beyond the scope of Sec. 5.8. 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. 5.3 and 5.4, soil-structure interaction will reduce the design values of the base shear and moment from the levels applicable to a rigid-base condition. These forces therefore can be evaluated conservatively without the adjustments recommended in Sec. 5.8.

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. 5.4 and involves the use of equivalent lateral static forces. The second is an extension of the simplified modal analysis procedure presented in Sec. 5.5. In the latter approach, the earthquake-induced effects are expressed as a linear combination of terms, the number of which is equal to the number of stories involved. Other more complex procedures also may be used, and these are outlined briefly at the end of this commentary on Sec. 5.8. 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.

5.8.2 Equivalent Lateral Force Procedure: This procedure is similar to that used in the older SEAOC recommendations except that it incorporates several improvements (see Sec. 5.4 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).

5.8.2.1 Base Shear: With the effects of soil-structure interaction neglected, the base shear is defined by Eq. 5.4.1, $V = C_s W$, in which W is the total dead weight of the structure and of applicable portions of the design live load (as specified in Sec. 5.4.1) and C_s is the dimensionless seismic response coefficient (as defined by Eq. 5.4.1.1-1 and 5.4.1.1-2). This term depends on the seismic zone 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 Eq. 5.4.1.1-1 and 5.4.1.1-2 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. 5.4.1 in the form:

$$V = C_s(T,\beta)\overline{W} + C_s(T,\beta)[W - \overline{W}] \qquad \text{(C5.8.2.1-1)}$$

where $\overline{W}$ represents the generalized or effective weight of the structure when vibrating in its fundamental natural mode. The terms in parentheses are used to emphasize the fact that C_s depends upon both T and β. The relationship between $\overline{W}$ and W is given below. The first term on the right side of Eq. C5.8.2.1-1 approximates the contribution of the fundamental mode of vibration whereas the second term approximates the contributions of the higher natural modes. Inasmuch as soil-structure interaction may be considered to affect only the contribution of the fundamental mode and inasmuch as this effect can be expressed by changes in the fundamental natural period and the associated damping of the system, the base shear for the interacting system, $\tilde{V}$, may be stated in a form analogous to Eq. C5.8.2.1-1:

$$\tilde{V} = C_s(\tilde{T},\tilde{\beta})\overline{W} + C_s(T,\beta)[W - \overline{W}] \qquad \text{(C5.8.2.1-2)}$$

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

Before proceeding with the evaluation of the coefficients C_s in Eq. C5.8.2.1-2, it is desirable to rewrite this formula in the same form as Eq. 5.8.2.1-1. Making use of Eq. 5.4.1 and rearranging terms, the following expression for the reduction in the base shear is obtained:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\tilde{\beta}) \right] \overline{W} \qquad \text{(C5.8.2.1-3)}$$

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 (e.g., T), are related approximately as follows:

$$C_s(\tilde{T},\tilde{\beta}) = C_s(\tilde{T},\beta) \left(\frac{\beta}{\tilde{\beta}} \right)^{0.4} \qquad \text{(C5.8.2.1-4)}$$

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

Substitution of Eq. C5.8.2.1-4 in Eq. C5.8.2.1-3 leads to:

$$\Delta V = \left[C_s(T,\beta) - C_s(\tilde{T},\beta) \left(\frac{\beta}{\tilde{\beta}} \right)^{0.4} \right] \overline{W} \qquad \text{(C5.8.2.1-5)}$$

where both values of C_s are now for the damping factor of the rigidly supported system and may be evaluated from Eq. 5.4.1.1-1 and 5.4.1.1-2. If the terms corresponding to the periods T and $\tilde{T}$ are denoted more simply as C_s and $\tilde{C}_s$, respectively, and if the damping factor β is taken as 0.05, Eq. C5.8.2.1-5 reduces to Eq. 5.8.2.1-2.

Note that $\tilde{C}_s$ in Eq. 5.8.2.1-2 is smaller than or equal to C_s because Eq. 5.4.1 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. 5.8.2.1.2-1), the shear reduction ΔV is a non-negative quantity. It follows that the design value of the base shear for the elastically supported structure cannot be greater than that for the associated rigid -base structure.

The effective weight of the structure, $\overline{W}$, is defined by Eq. 5.5.4-2 (Sec. 5.5), in which ϕ_{im} should be interpreted as the displacement amplitude of the i^{th} floor when the structure is vibrating in its fixed-base fundamental natural mode. It should be clear that the ratio $\overline{W} / W$ depends on the detailed characteristics of the structure. A constant value of $\overline{W} = 0.7\ W$ 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 $\overline{W} = 0.75\ W$. Naturally, when the full weight of the structure is concentrated at a single level, $\overline{W}$ 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.

5.8.2.1.1 Effective Building Period: Equation 5.8.2.1.1-1 for the effective natural period of the elastically supported structure, $\tilde{T}$, is determined from analyses in which the superstructure is presumed to respond in its fixed-base fundamental mode and the foundation weight is considered to be negligible in comparison to the weight of the superstructure (Jennings and Bielak, 1973; Veletsos and Meek, 1974). The first term under the radical represents the period of the fixed-base structure. The first portion of the second term represents the contribution to $\tilde{T}$ of the translational flexibility of the foundation, and the last portion represents the contribution of the corresponding rocking flexibility. The quantities $\bar{k}$ and $\bar{h}$ represent, respectively, the effective stiffness and effective height of the structure, and K_y and K_θ represent the translational and rocking stiffnesses of the foundation.

Equation 5.8.2.1.1-2 for the structural stiffness, $\bar{k}$, is deduced from the well known expression for the natural period of the fixed-base system:

$$T = 2\pi\sqrt{\left(\frac{1}{g}\right)\left(\frac{\overline{W}}{\bar{k}}\right)} \qquad \text{(C5.8.2.1.1-1)}$$

The effective height, $\bar{h}$, is defined by Eq. 5.8.3.1-2, in which ϕ_{il} has the same meaning as the quantity ϕ_{im} in Eq. 5.5.4.-2 when $m = 1$. In the interest of simplicity and consistency with the approximation used in the definition of $\overline{W}$, 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:

$$K_y = \left[\frac{8\alpha_y}{2 - v}\right]Gr \qquad \text{(C5.8.2.1.1-2)}$$

and

$$K_\theta = \left[\frac{8\alpha_\theta}{3(1-v)}\right] Gr^3 \qquad \text{(C5.8.2.1.1-3)}$$

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:

$$G = \frac{\gamma v_s^2}{g} \qquad \text{(C5.8.2.1.1-4)}$$

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. C5.8.2.1.1-2 and 5.8.2.1.1-3 reduce to:

$$K_y = \frac{8Gr}{2 - v} \qquad \text{(C5.8.2.1.1-5)}$$

and

$$K_\theta = \frac{8Gr^3}{3(1 - v)} \qquad \text{(C5.8.2.1.1-6)}$$

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. C5.8.2.1.1-5. 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. 5.8.2.1.2-3.

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 $\bar{h}/r$ ~ 0.5 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:

$$K_y = \left[\frac{8Gr}{2 - v}\right]\left[1 + \left(\frac{2}{3}\right)\left(\frac{d}{r}\right)\right] \tag{C5.8.2.1.1-7}$$

and

$$K_\theta = \left[\frac{8Gr^3\alpha_\theta}{3(1 - v)}\right]\left[1 + 2\left(\frac{d}{r}\right)\right] \tag{C5.8.2.1.1-8}$$

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. C5.8.2.1.1-7 and C5.8.2.1.1-8. For example, if a structure is embedded in such a way that there is no positive contact between the soil and the walls of the structure, or when any existing contact cannot reasonably be expected to remain effective during the stipulated design ground motion, stiffnesses K_y and K_θ should be determined from the formulas for surface-supported foundations. More generally, the quantity d in Eq. C5.8.2.1.1-7 and C5.8.2.1.1-8 should be interpreted as the effective depth of foundation embedment for the conditions that would prevail during the design earthquake.

The formulas for K_y and K_θ presented above are strictly valid only for foundations supported on reasonably uniform soil deposits. When the foundation rests on a 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. C5.8.2.1.1-7:

$$K_y \simeq \left[\frac{8Gr}{2 - \nu}\right]\left[1 + \left(\frac{2}{3}\right)\left(\frac{d}{r}\right)\right]\left[1 + \left(\frac{1}{2}\right)\left(\frac{r}{D_s}\right)\right]\left[1 + \left(\frac{5}{4}\right)\left(\frac{d}{D_s}\right)\right] \quad \text{(C5.8.2.1.1-9)}$$

Second, using Eq. C5.8.2.1.1-8:

$$K_\theta \simeq \left[\frac{8Gr^3\alpha_\theta}{3(1 - \nu)}\right]\left[1 + 2\left(\frac{d}{r}\right)\right]\left[1 + \left(\frac{1}{6}\right)\left(\frac{r}{D_s}\right)\right]\left[1 + 0.7\left(\frac{d}{D_s}\right)\right] \quad \text{(C5.8.2.1.1-10)}$$

These formulas are based on analyses of a stratum supported on a rigid base (Elsabee et al.,1977; Kausel and Roesset, 1975) 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. 5.8.2.1.1-5), 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. 5.8.2.1.1-6), 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:

$$K_y = \Sigma k_{yi} \quad \text{(C5.8.2.1.1-11)}$$

and

$$K_\theta = \Sigma k_{xi} y_i^2 + \Sigma k_{\theta i} \quad \text{(C5.8.2.1.1-12)}$$

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

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

$$k_{yi} = \left(\frac{8G_i r_{ai}}{2 - \nu}\right)\left(1 + \frac{2}{3}\frac{d}{r}\right) \tag{C5.8.2.1.1-13}$$

$$k_{xi} = \left(\frac{4G_i r_{ai}}{1 - \nu}\right)\left(1 + 0.4\frac{d}{r}\right) \tag{C5.8.2.1.1-14}$$

and

$$k_{\theta i} = \left[\frac{8G_i r_{mi}^3}{3(1 - \nu)}\right]\left[1 + 2\frac{d}{r}\right] \tag{C5.8.2.1.1-15}$$

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

For surface-supported footings and for embedded footings for which the side wall contact with the soil cannot be considered to be effective during the stipulated design ground motion, d_i in these formulas should be taken as zero. Furthermore, the values of G_i should be consistent with the stress levels expected under the footings and should be evaluated with due regard for the effects of the dead loads involved. This matter is considered further in subsequent sections. For closely spaced footings, consideration of the coupling effects among footings will reduce the computed value of the overall foundation stiffness. This reduction will, in turn, increase the fundamental natural period of the system, $\tilde{T}$, and 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. C5.8.2.1.1-11 and 5.8.2.1.1-12 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. C5.8.2.1.1-11 and 5.8.2.1.1-12.

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, ν. These quantities are likely to vary from point to point of a construction site, and it is necessary to use average values for the soil region that is affected by the forces acting on the foundation. The depth of significant influence is a function of the dimensions of the foundation base and of the direction of the motion involved. The effective depth may be considered to extend to about $0.75r_a$ 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 5.8.2.1.1. The backgrounds of this relationship and of the corresponding relationship for v_s/v_{so} are identified below.

The low amplitude value of the shear modulus, G_o, can most conveniently be determined from the associated value of the shear wave velocity, v_{so}, by use of Eq. C5.8.2.1.1-4. The latter value may be determined approximately from empirical relations or more accurately by means of field tests or laboratory tests.

The quantities G_o and v_{so} depend on a large number of factors (Hardin, 1978), the most important of which are the void ratio, e, and the average confining pressure, $\overline{\sigma}_o$. The value of the latter pressure at

$$\overline{\sigma}_o = \overline{\sigma}_{os} + \overline{\sigma}_{ob} \qquad \text{(C5.8.2.1.1-16)}$$

a given depth beneath a particular foundation may be expressed as the sum of two terms as follows:

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

$$\overline{\sigma}_{os} = \left(\frac{1 + 2K_o}{3} \right) \gamma' x \qquad \text{(C5.8.2.1.1-17)}$$

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 $\overline{\sigma_{ob}}$ developed by the weight of the structure can be estimated from the theory of elasticity (Poulos and Davis, 1974). In contrast to $\overline{\sigma_{os}}$ which increases linearly with depth, the pressures $\overline{\sigma_{ob}}$ decrease with depth. As already noted, the value of v_{so} should correspond to the average value of $\overline{\sigma_o}$ 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

$$v_{so} = c_1(2.17 - e)(\overline{\sigma})^{0.25} \qquad \text{(C5.8.2.1.1-18)}$$

approximately from the formula:

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

$$v_{so} = c_2(2.97 - e)(\overline{\sigma})^{0.25} \qquad \text{(C5.8.2.1.1-19)}$$

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

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

Equation C5.9.2.1.1-19 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:

$$G_o = 1{,}000 S_u \qquad \text{(C5.8.2.1.1-20)}$$

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 are that v_{so} is determined only for the stress conditions existing at the time of the test (usually $\overline{\sigma}_{so}$). The effect of the changes in the stress conditions caused by construction must be considered by use of

Eq. C5.8.2.1.1-17 and Eq. C5.8.2.1.1-18 and C5.8.2.1.1-19 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 5.8.2.1.1. 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 5.8.2.1.1. For each severity of anticipated ground-shaking, represented by the effective peak acceleration coefficients A_a and A_v, a representative value of shearing strain amplitude was developed. A conservative value of v_s/v_{so} that is appropriate to that strain amplitude then was established. It should be emphasized that the values in Table 5.8.2.1.1 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: $\nu = 0.33$ for clean sands and gravels, $\nu = 0.40$ for stiff clays and cohesive soils, and $\nu = 0.45$ for soft clays. The use of an average value of $\nu = 0.4$ also will be adequate for practical purposes.

Regarding an alternative approach, note that Eq. 5.8.2.1.1-3 for the period $\tilde{T}$ of structures supported on mat foundations was deduced from Eq. 5.8.2.1.1-1 by making use of Eq. C5.8.2.1.1-5 and C5.8.2.1.1-6, with Poisson's ratio taken as $\nu = 0.4$ and with the radius r interpreted as r_a in Eq. C5.8.2.1.1-5 and as r_m in Eq. C5.8.2.1.1-6. For a nearly square foundation, for which $r_a \simeq r_m \simeq r$, Eq. 5.8.2.1.1-3 reduces to:

$$\tilde{T} = T\sqrt{1 + 25\alpha\left(\frac{r\bar{h}}{v_s^2 T^2}\right)\left[1 + \left(\frac{1.12\overline{h^2}}{\alpha_\theta r^2}\right)\right]} \qquad \text{(C5.8.2.1.1-21)}$$

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

5.8.2.1.2 Effective Damping: Equation 5.8.2.1.2-1 for the overall damping factor of the elastically supported structure, $\tilde{\beta}$, was determined from analyses of the harmonic response at resonance of simple systems of the type considered in Figures C5.8.1-2 and 5.8.1-3. The result is an expression of the form (Bielak, 1975; Veletsos and Nair, 1975):

$$\tilde{\beta} = \beta_o + \frac{0.05}{\left(\frac{\tilde{T}}{T}\right)^3} \qquad \text{(C5.8.2.1.2-1)}$$

in which β_o represents the contribution of the foundation damping, considered in greater detail in the following paragraphs, and the second term represents the contribution of the structural damping. The latter damping is assumed to be of the viscous type. Equation C5.8.2.1.2-1 corresponds to the value of $\beta = 0.05$ used in the development of the response spectra for rigidly supported systems employed in Sec. 5.4.

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

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

The variation of β_o with $\tilde{T}/T$ and $\bar{h}/r$ is given in Figure 5.8.2.1.2 for two levels of excitation. The dashed lines, which are recommended for values of the design earthquake spectral response acceleration at short periods, S_{DS} , equal to or less than 0.25, correspond to a value of $\Delta W_s/W_s \simeq 0.3$, whereas the solid lines, which are recommended for S_{DS} values 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 5.8.2.1.2 should be interpreted as a characteristic length that is related to the length of the foundation, L_o, in the direction in which the structure is being analyzed. For short, squatty structures for which $\bar{h}/L_o \leq 0.5$, 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. 5.8.2.1.1-5). On the other hand, for structures with $\bar{h}/L_o \geq 1$, the interaction effects are dominated by the rocking motion of the foundation, and it is reasonable to define r as the radius r_m of a disk whose static moment of inertia about a horizontal centroidal axis is the same as that of the actual foundation normal to the direction in which the structure is being analyzed (see Eq. 5.8.2.1.1-6).

Subject to the qualifications noted in the following section, the curves in Figure 5.8.2.1.2 also may be used for embedded mat foundations and for foundations involving spread footings or piles. In the latter cases, the quantities A_o and I_o in the expressions for the characteristic foundation length, r, should be interpreted as the area and the moment of inertia of the load-carrying foundation.

In the evaluation of the overall damping of the structure-foundation system, no distinction has been made between surface-supported foundations and embedded foundations. Since the effect of embedment is to increase the damping capacity of the foundation (Bielak, 1975; Novak, 1974; Novak and Beredugo, 1972) and since such an increase is associated with a reduction in the magnitude of the forces induced in the structure, the use of the recommended 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. 5.8 and in the requirements for rigidly supported structures presented in Sec. 5.4 and 5.5.

Equations 5.8.2.1.2-1 and C5.8.2.1.2-2, in combination with the information presented in Figure 5.8.2.1.2, may lead to damping factors for the structure-soil system, $\tilde{\beta}$, that are smaller than the structural damping factor, β. However, since the representative value of $\beta = 0.05$ used in the development of the design 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. 5.8.2.1.2-1 should never be taken less than β, and a low bound of $\tilde{\beta} = \beta = 0.05$ has been imposed. The use of values of $\tilde{\beta} > \beta$ is justified by the fact that the experimental values correspond to extremely small amplitude motions and do not reflect the effects of the higher soil damping capacities corresponding to the large soil strain levels associated with the design ground motions. The effects of the higher soil damping capacities are appropriately reflected in the values of β_o presented in Figure 5.8.2.1.5.

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,

$$T_s = \frac{4D_s}{v_s} \tag{C5.8.2.1.2-2}$$

is smaller than the natural period of the flexibly supported structure, $\tilde{T}$. The quantity D_s in this formula represents the depth of the stratum. It follows that the values of β_o presented in Figure 5.8.2.1.2 are applicable only when:

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} \geq 1 \qquad \text{(C5.8.2.1.2-3)}$$

for

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} \leq 1 \qquad \text{(C5.8.2.1.2-4)}$$

$$\frac{T_s}{\tilde{T}} = \frac{4D_s}{v_s\tilde{T}} < 1 \qquad \text{(C5.8.2.1.2-4)}$$

the effective value of the foundation damping factor, β_o', is less than β_o, and it is approximated by the second degree parabola defined by Eq. 5.8.2.1.2-4.

For $T_s/\tilde{T} = 1$, Eq. 5.8.2.1.2-4 leads to $\beta_o' = \beta_o$ whereas for $T_s/\tilde{T} = 0$, it leads to $\beta_o' = 0$, a value that clearly does not provide for the effects of material soil damping. It may be expected, therefore, that the computed values of β_o' corresponding to small values of $T_s/\tilde{T}$ will be conservative. The conservatism involved, however, is partly compensated by the requirement that $\tilde{\beta}$ be no less than $\tilde{\beta} = \beta = 0.05$.

5.8.2.2 and 5.8.2.3 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. Methods exist for incorporating two- and three-dimensional *P*-delta effects into computer analyses that do not explicable 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 in the ATC 3-06 document, however, there has been some debate regarding its accuracy. This debate reflects the intuitive notion that a structure's secant stiffness would more accurately represent inelastic *P*-delta effects. Due to the additional uncertainty of the effect of dynamic response on *P*-delta behavior and on the (apparent) observation that instability-related failures rarely occur in real structures, the *P*-delta requirements as originally written have remained unchanged until now.

There is increasing evidence, however, that the use of inelastic stiffness in determining theoretical *P*-delta response is unconservative. Based on a study carried out by Bernal (1987), it can be argued that *P*-delta amplifiers should be based on secant stiffness. In other words, the C_d term in Eq. 5.4.6.2.-1 of the *Provisions* should be deleted. Since Bernal's study was based on the inelastic dynamic response of single-degree-of-freedom elastic-perfectly plastic systems, significant uncertainties exist in the extrapolation of the concepts to the complex hysteretic behavior of multi-degree-of-freedom systems.

Another problem with accepting a *P*-delta procedure based on secant stiffness is that current design forces would be greatly increased. For example, consider an ordinary moment frame of steel with a C_d of 4.0 and an elastic stability coefficient, θ, of 0.15. The amplifier for this structure would be 1.0/0.85 = 1.18 according to the current requirements. 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 = 5.50. (Note that the 0.9 in the numerator of the amplifier equation in the 1988 Edition of the *Provisions* has been dropped for this comparison.) From this example, it can be seen that there could be an extreme impact on the requirements if a change was implemented that incorporated *P*-delta amplifiers based on static secant stiffness response.

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

1. Many structures display an overstrength well above the strength implied by code-level design forces (see Figure 5.8.1). This overstrength likely protects structures from stability-related failures.

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

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

Aside from the amplifier, however, the 1991 Edition of the *Provisions* added a new requirement that the computed stability coefficient, θ, not exceed 0.25 or $0.5/\beta C_d$ where βC_d is an adjusted ductility demand that takes into account the fact that the seismic strength demand may be somewhat less than the code strength supplied. The adjusted ductility demand is not intended to incorporate overstrength beyond that computed by the means available in Chapters 8 though 14 of the *Provisions*.

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

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

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

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

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

$$\overline{W} = \overline{W_1} = \frac{\left(\Sigma w_i \phi_{il}\right)^2}{\Sigma w_i \phi_{il}^2} \qquad \text{(C5.8.3)}$$

which is the same as Eq. 5.5.4-2, and $\overline{h}$ is computed from Eq. 5.8.3.1-2. The quantity ϕ_{il} 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, $\overline{k}$, is obtained from Eq. 5.8.2.1.1-2 by taking $\overline{W} = \overline{W}_1$ and using for T the fundamental natural period of the fixed-base structure, T_1. The fundamental natural period of the interacting system, $\tilde{T}_l$, is then computed from Eq. 5.8.2.1.1-1 (or Eq. 5.8.2.2.1.1-3 when applicable) by taking $T = T_1$. The effective damping in the first mode, β, is determined from Eq. 5.8.2.1.2-1 (and Eq. 5.8.2.1.2-4 when applicable) in combination with the information given in Figure 5.8.2.1.2. The quantity $\bar{h}$ in the latter figure is computed from Eq. 5.8.3.1-2.

With the values of $\tilde{T}_l$ and $\tilde{\beta}_l$ established, the reduction in the base shear for the first mode, ΔV_l, is computed from Eq. 5.8.2.1-2. The quantities C_s and $\tilde{C}_s$ in this formula should be interpreted as the seismic coefficients corresponding to the periods T_1 and $\tilde{T}_1$, respectively; $\tilde{\beta}$ should be taken equal to $\tilde{\beta}_l$; and $\overline{W}$ should be determined from Eq. C5.8.3.

The sections on lateral forces, shears, overturning moments, and displacements follow directly from what has already been noted in this and the preceding sections and need no elaboration. It may only be pointed out that the first term within the brackets on the right side of Eq. 5.8.3.2-1 represents the contribution of the foundation rotation.

5.8.3.3 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. 5.4.4 and the *P*-delta effects should be evaluated in accordance with the requirements of Sec. 5.4.6.2, using the story shears and drifts determined in Sec. 5.8.3.2.

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_o, by considering in a more precise manner the foundation type involved, the effects of foundation embedment, variations of soil properties with depth, and hysteretic action in the soil. Solutions may be obtained in some cases with analytical or semi-analytical formulations and in others by application of finite difference or finite element techniques. 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 pre-dominant soil strains induced by the design ground motion.

3. Incorporate the effects of interaction for the higher modes of vibration of the structure, either approximately by application of the procedures recommended in Bielak (1976), Roesset et al. (1973), and Tsai (1974) or by more precise analyses of the structure-soil system. The latter analyses may be implemented either in the time domain by application of the impulse response functions presented in Veletsos and Verbic (1974). However, the frequency domain analysis is limited to systems that respond within the elastic range while the approach involving the use of the impulse response functions is limited, at present, to soil deposits that can adequately be represented as a uniform elastic halfspace. The effects of yielding in the structure and/or supporting medium can be considered only approximately in this approach by representing the supporting medium by a series of springs and dashpots whose properties are independent of the frequency of the motion and by integrating numerically the governing equations of motion (Parmelee et al., 1969).
4. Analyze the structure-soil system by finite element method (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 ANALYSIS PROCEDURE

C5A.1 NONLINEAR STATIC ANALYSIS PROCEDURE: The analysis 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 through measurement of the strength of the structure, at various increments of lateral displacement. The strength is measured by the shear forces resisted by a structure in the form of lateral forces, which cause the lateral deformations.

Usually the shear resisted by the system when the first element yields in the structure, although not always relevant for the entire structure, is defined as the "elastic strength." When traditional linear methods of design are used, together with R factors, the value of the design base shear sets the minimum strength at which this elastic strength point can occur.

If a structure is subjected to larger lateral loads, then represented by the elastic strength, than 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 than the mechanism will not allow increase of deformations without some increase of lateral forces and the mechanisms is stable. Moreover, it can be considered as a flexible version of the original frame structure. Figure C5A.7-1, which shows a plot of lateral structural strength vs. deformation of a hypothetical structure, sometimes termed a pushover curve, illustrates these concepts.

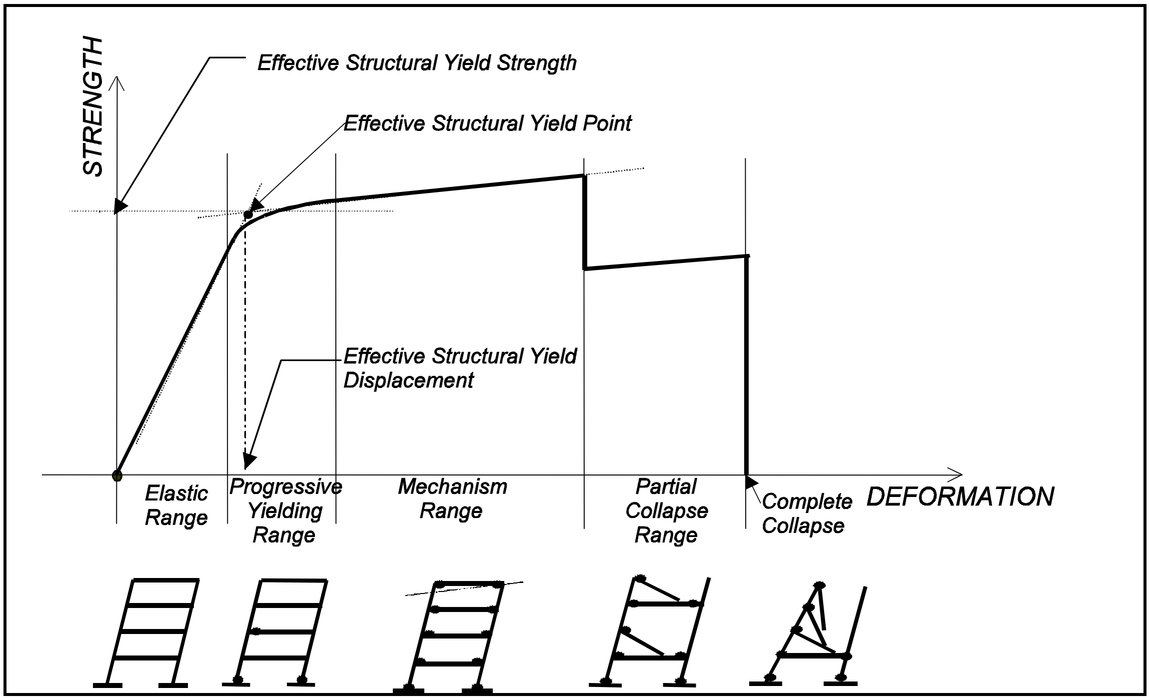

FIGURE C5A.7.1 Strength deformation relation in a frame structure.

If after the structure develops a mechanism it deforms an additional substantial amount, elements within the structure may fail, fracture, or buckle, etc., losing their strength contribution to the whole structural system. In such case, the strength of the structure will diminish with increasing deformation. If any essential element, or group of elements, fail, then the entire structure may loose capacity to carry the gravity loads, or any lateral load. This condition can also occur if the lateral deformation becomes so great that the P-delta effects exceed the residual lateral strength of the structure. Such conditions are defined as collapse and the deformation associated with collapse 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 C5A.7-1, 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 analysis is to provide a simplified method of determining structural response behavior at deformation levels intermediate to those which can be conveniently analyzed using limit state methods.

C5A.1.1 Modeling: In performing this method, the structure is modeled with elements having stiffness properties that are dependent on the amount of deformation imposed on the element. All elements than can experience deformations or forces larger than yield should be modeled with nonlinear properties. As a minimum, nonlinear stiffness properties should be described, by a bilinear model, with 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 defined.

C5A.1.1.2 Lateral Loads: The analysis is performed by applying a monotonically increasing "set of loads" distributed throughout the structure. The analysis traces the internal distribution loads and deformations as the set of loads 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 C5A.7-1.

It should be noted that nonlinear static analysis can determine the order of yielding of elements in the "progressive yielding range" (see Figure C5A.7-1) and the associated strength and deformations. The analysis can also determine the deformations associated with fractures or failure of *components* and the entire structure. However, it is accurate, only if the applied set 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 participation in higher modes.

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, changing the load distribution at each step, accordingly.

Such use of a varying, deformation-dependent load configuration would require almost as much labor and uncertainties 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:

(a) An approximate distribution proportional to the idealized elastic response model as used in the equivalent lateral force method:

$$F_i = \frac{W_i h_i^k}{\sum_j W_j h_i^k} V \qquad \text{(C5A.1.2-1)}$$

where, F, W, h and V are the story inertia force, the story weight and height, and the base shear, respectively; k is a power index ranging between 1 and 2 as defined in ATC3-06.

(b) A better approximation is obtained if the dominant mode of vibration is known, such as the first mode in moderate height building structures:

$$F_i = \frac{W_i \varphi_i}{\sum_i w_i \varphi_i} V \qquad \text{(C5A.1.2-2)}$$

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.

(c) A still more complete approximation can be obtained, if several significant modes of vibration are also known. In such cases the modes for which the total equivalent modal mass exceed 90 percent should be included. The load configuration is given by:

$$F_i = V\frac{W_i\varphi_{id}}{\Sigma w_i\varphi_{id}}\frac{\left[\Sigma\left[\left(\Gamma_i/\Gamma_d\right)\left(S_{ai}/S_{ad}\right)\right]^2\right]^{1/2}}{\left[\Sigma\left[\left(\Gamma_i/\Gamma_d\right)^2\left(S_{ai}/S_{ad}\right)\right]^2\right]^{1/2}} \qquad \text{(C5A.1.2-3)}$$

where, Γ_i/S_{ai} are the modal participation factor and the spectral acceleration, respectively, and subscript d indicates the dominant mode. ($\Gamma_t = \Sigma W_i\varphi_i$; where the mode shapes are ϕ are mass normalized, i.e. $\Sigma W_i\varphi_i^2 / g = 1$).

(d) If more accurate definition of the load is necessary then the configuration described by Eq. (C5A.1.2-3) should be calculated and reevaluated when changes occur in the modal characteristics of the structure as it yields. Such procedure has also defined as “adaptablepush-over.”

The *Provisions* adopt the simplest of these approaches, indicated as (a) above, though the use of the more complex approaches should not be preluded. Nonlinear static analysis in several commercially available and public domain nonlinear analysis platforms.

C5A.1.3 Limit Deformation: The nonlinear analysis should be continued by increasing the loading set until the deflections at the control point exceeds 150 percent of the expected inelastic deflection. The expected inelastic deflection at each level shall be determined by combining the elastic modal values as obtained from Sec. 5.5.5 and 5.5.6 multiplied by the factor

$$C_i = \frac{\left(1 - T_s/T_1\right)}{R_d} + \left(T_s/T_1\right) \qquad \text{(C5A.1.3-1)}$$

where T_s is the characteristic period of the response spectrum, defined as the period associated to 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 5.2.2. 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 the 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 the epicenter distance, yield level, strain-hardening ratio ans the stiffness degradation was examined. The study concluded that epicentral distance and stiffness degradation have negligible influence on strength reduction factors. Ratios of inelastic displacements to displacements predicted by elastic analysis were derived from the above work:

$$R_d = \left[1 + \frac{1}{c}\left(r^c - 1\right)\right] / r \cdot \geq 1 \qquad \text{(C5A.1.3-1)}$$

$$c = \frac{T^a}{1 + T^a} + \frac{b}{T} \qquad \text{(C5A.1.3-2)}$$

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

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

$$R_D = \left(1 - \frac{1}{r}\right)\left(\frac{T_{PV}}{T}\right)^n + \frac{1}{r} \cdot \geq 1 \qquad \text{(C5A.1.3-3)}$$

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

n=1.2+0.025r for $T_{PV} \leq 1.2\,\sec$ (C5A.1.3-4.a)

n=1.2 for $T_{PV} > 1.2\,\sec$ (C5A.1.3-4.b)

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.

$$R_D = \left(1 - \frac{1}{r}\right)\frac{T_0}{T} + \frac{1}{r} \cdot \geq 1 \qquad \text{(C5A.1.3-5)}$$

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

$$T_1 = 2\pi \frac{\varphi_{ev}}{\varphi_{ea}} \frac{V}{A} \qquad \text{(C5A.1.3-6)}$$

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); on the other hand, magnitude and epicenter distance have a negligible effect on mean strength reduction factors.

$$R_D = \left(1 - \frac{1}{r}\right)\Phi + \frac{1}{r} \cdot \qquad \text{(C5A.1.3-7)}$$

$$\Phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T}\exp\left[-\frac{3}{2}\left(\ln T - \frac{3}{5}\right)^2\right] \qquad \text{(C5A.1.3-8)}$$

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T}\exp\left[-2\left(InT - \frac{1}{5}\right)^2\right] \qquad \text{(C5A.1.3-9)}$$

$$\Phi = 1 + \frac{T_g}{3T} - \frac{3T_g}{2T}\exp\left[-3\left(\ln\frac{T}{T_g} - \frac{1}{4}\right)^2\right] \qquad \text{(C5A.1.3-10)}$$

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 inaccuracy is covered by the request of 50 percent accedence of the calculated target. In addition the 50 percent margin is required since a small variation in strength (due to modeling or due to imprecise construction) can lead to large displacement variations in the inelastic range.