3.3.1.1.1 Supplemental Damping

If damping can be significantly increased, then structural responses (and therefore forces and displacements) are greatly reduced (Hanson et al. 1993). Supplemental damping systems include *friction* systems (e.g., Sumitomo, pall, and friction-slip) based on Coulomb friction, *self-centering* friction resistance that is proportional to displacement (e.g., Fluor–Daniel energy dissipating restraint), or various *energy dissipation* mechanisms: ADAS (added damping and stiffness) elements, which utilize the yielding of mild-steel X-plates; viscoelastic shear dampers using a 3M acrylic copolymer as the dissipative element; or nickel–titanium alloy shape-memory devices that take advantage of reversible, stress-induced phase changes in the alloy to dissipate energy (Aiken et al. 1993). These systems are generally still in the developmental stage although there are no special obstacles for the implementation of the ADAS system, which has seen one application to date in the United States (Perry et al. 1993) and more in other countries.

3.3.1.2 Active Control

Active control depends on actively modifying a structure's mass, stiffness, or geometric properties during its dynamic response in such a manner so as to counteract and reduce excessive displacements (Iemura and Pradono 2002). Tuned mass dampers, reliance on liquid sloshing (Lou et al. 1994), and active tensioning of tendons are methods currently under investigation. Most methods of active control are real time, relying on measurement of structural response, rapid structural computation, and fast-acting energy sources. A number of issues of reliability remain to be resolved (Spencer et al. 1994).

3.4 Types of Buildings and Typical Earthquake Performance

There are many different types of buildings, with varying kinds of earthquake performance and seismic design needs. This section discusses general earthquake performance of buildings, with the emphasis more toward those buildings typically built in the western United States. Specific aspects of structural analysis and design of buildings, other structures, steel, concrete, wood, masonry, and other topics are discussed in other chapters.

In buildings, earthquake performance can be divided into two categories: structural and non-structural, both of which when unsatisfactory can be hazardous to building occupants — when *damage* occurs. Structural damage means degradation of the building's structural support systems (i.e., vertical and lateral force resisting systems), such as the building frames and walls. Nonstructural damage refers to any damage that does not affect the integrity of the structural support system. Examples of nonstructural damage are a chimney collapsing, windows breaking, ceilings falling, piping damage, and disruption of pumps, control panels, telecommunications equipment, etc. Nonstructural damage can still be life threatening and costly. The type of damage to be expected is a complex issue that depends on the structural type and age of the building, its configuration, construction materials, the site conditions, the proximity of the building to neighboring buildings, and the type of nonstructural elements.

The typical earthquake performances of different types of common building structural systems are described in this section to provide insights into seismic design for buildings.

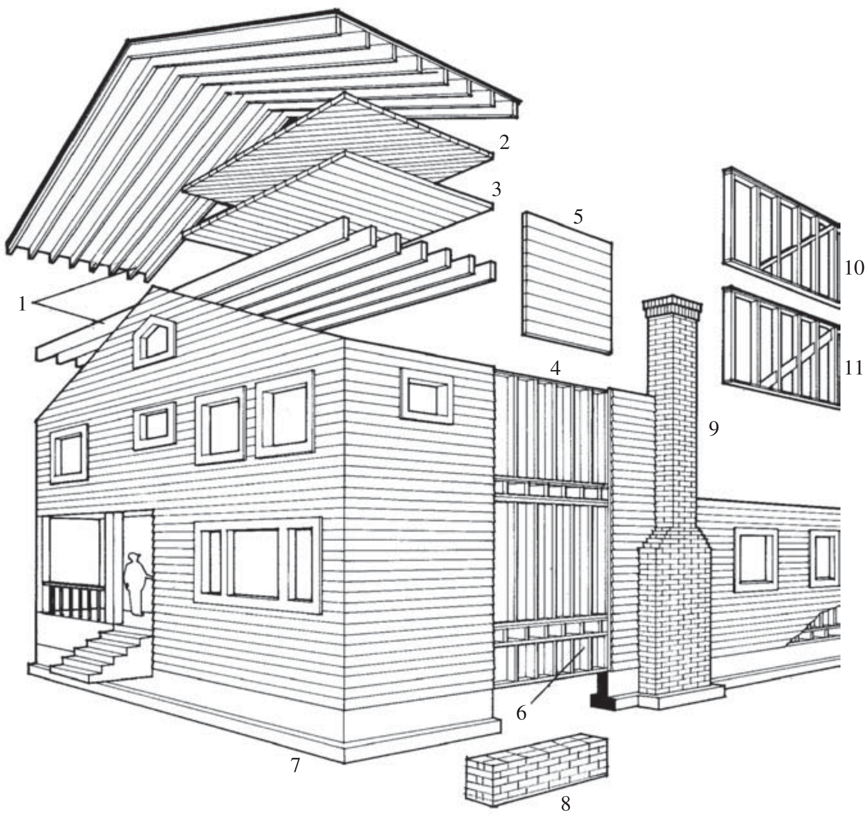
3.4.1 Wood Frame

Wood-frame structures tend to be mostly low rise (one to three stories, occasionally four stories). Vertical framing may be of several types, for example, stud wall, braced post and beam, or timber pole:

- Stud walls are typically constructed of 2 in. by 4 in. wood members vertically set about 16 in. apart — multiple story buildings may have 2×6 or larger studs. These walls are braced by plywood sheathing or by diagonals made of wood or steel. Most detached single and low-rise multiple family residences in the United States are of stud wall wood frame construction, Figure 3.4.

Roof/floor span systems
1. Wood joist and rafter
2. Diagonal sheathing
3. Straight sheathing

Wall systems
4. Stud wall (platform or balloon frame)
5. Horizontal siding



Foundation/connections
6. Unbraced cripple wall
7. Concrete foundation
8. Brick foundation

Bracing and details
9. Unreinforced brick chimney
10. Diagonal blocking
11. Let-in brace (only in vintage)

FIGURE 3.4 Schematic of wood light-frame construction. (From Federal Emergency Management Agency. 1988. *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, FEMA 154, FEMA, Washington, DC.)

- Post and beam construction is not very common in the United States, although it is the basis of the traditional housing in other countries (e.g., Europe, Japan); in the United States, it is found mostly in older housing and larger buildings (i.e., warehouses, mills, churches, and theaters). This type of construction consists of larger rectangular (6 in. by 6 in. and larger) or sometimes round wood columns framed together with large wood beams or trusses.
- Timber pole buildings are a less common form of construction found mostly in suburban/rural areas. Generally adequate seismically when first built, they are more often subject to wood deterioration due to the exposure of the columns, particularly near the ground surface. Together with an often-found "soft story" in this building type, this deterioration may contribute to unsatisfactory seismic performance.

In wood frame stud-wall buildings, the resistance to lateral loads is typically provided by (a) for older buildings, especially houses, wood diagonal "let-in" bracing and (b) for newer (primarily post-World War II) buildings, plywood siding "shear walls." Without the extra strength provided by the bracing or plywood, walls would distort excessively or "rack," resulting in broken windows, stuck doors, cracked plaster, and, in extreme cases, collapse.

Stud-wall buildings have performed very well in past U.S. earthquakes for ground motions of about 0.5*g* or less, due to inherent qualities of the structural system and because they are lightweight and low rise. Cracking in plaster and stucco may occur and these act to degrade the strength of the building to some extent (i.e., the plaster and stucco may in fact form part of the LFRS, sometimes by design) — this is usually classified as nonstructural damage but, in fact, dissipates a lot of the earthquake-induced energy. However, the most common type of structural damage in older wood-frame buildings results from a lack of connection between the superstructure and the foundation — the so-called "cripple wall" construction. This kind of construction is common in the milder climes of the west, where full basements are not required, and consists of an air space (typically 2–3 ft) left under the house — the short stud walls under the first floor (termed by carpenters a cripple wall because of their less than full height) were usually built without bracing so that their is no adequate LFRS for this short height. Plywood sheathing nailed to the cripple studs may have been used to strengthen the cripple walls. Additionally, the mud sill in these older (typically pre-World War II) housing may not be bolted to the foundation. As a result, houses can slide off their foundations when not properly bolted to the foundation, resulting in major damage to the building as well as to plumbing and electrical connections. Overturning of the entire structure is usually not a problem because of the low-rise geometry. In many municipalities, modern codes require wood structures to be bolted to their foundations. However, the year that this practice was adopted will differ from community to community and should be checked.

Garages often have a very large door opening in one wall with little or no bracing. This wall has almost no resistance to lateral forces, which is a problem if a heavy load such as a second story sits on top of the garage (the so-called house over garage, or HOGs). Homes built over garages have sustained significant amounts of damage in past earthquakes, with many collapses. Therefore, the HOG configuration, which is found commonly in low-rise apartment complexes and some newer suburban detached dwellings, should be examined more carefully and perhaps strengthened.

Unreinforced masonry (URM) chimneys also present a life-safety problem. They are often inadequately tied to the building and therefore fall when strongly shaken. On the other hand, chimneys of reinforced masonry generally perform well.

Some wood-frame structures, especially older buildings in the eastern United States, have masonry veneers that may represent another hazard. The veneer usually consists of one wythe of brick (a wythe is a term denoting the width of one brick) attached to the stud wall. In older buildings, the veneer is either insufficiently attached or has poor quality mortar, which often results in peeling off of the veneer during moderate and large earthquakes.

Post and beam buildings tend to perform well in earthquakes if adequately braced. However, walls often do not have sufficient bracing to resist horizontal motion and thus they may deform excessively.

The 1994 M_W 6.7 Northridge earthquake was the largest earthquake to occur directly within an urbanized area since the 1971 San Fernando earthquake — ground motions were as high as 0.9*g* and substantial numbers of modern wood-frame dwellings sustained significant damage, including major cracking of veneers, gypsum board walls, and splitting of wood wall studs. It may be inferred from this, as well as the performance observed in the more sparsely populated epicentral regions of the 1989 M_W 7.1 Loma Prieta Earthquake, that U.S. single family dwelling design begins to sustain substantial nonstructural and structural damage for peak ground acceleration in excess of about 0.5*g*.

3.4.2 Steel-Frame Buildings

Steel-frame buildings generally may be classified as MRFs, braced frames, or mixed construction (e.g., steel frame for vertical forces and reinforced concrete shear wall for the LFRS) based on their LFRSs. In concentric braced frames the lateral forces or loads are resisted by the tensile and compressive strength of the bracing, which can assume a number of different configurations including diagonal, "V," inverted "V" also termed chevron, "K," etc. A recent development in seismic bracing is the eccentric brace frame. Here, the bracing is slightly offset from the main beam to column connection, and the short section of the beam is expected to deform significantly under major seismic forces and thereby dissipate a considerable portion of the energy. MRFs resist lateral loads and deformations by the bending stiffness of the beams and columns (there is no bracing), Figure 3.5.

Steel-frame buildings have tended to perform satisfactory in earthquakes with ground motions less than about 0.5*g* because of their strength, flexibility, and lightness. Collapse in earthquakes has been very rare, although steel-frame buildings did collapse, for example, in the 1985 Mexico City Earthquake. More recently, following the 1994 M_W 6.7 Northridge Earthquake, a number of MRFs were found to have sustained serious cracking in the beam column connection; see Figure 3.6, which shows one of a number of different types of cracking that were found following the Northridge Earthquake. The cracking typically initiated at the lower beam flange location and propagated upward into the shear panel. Similar cracking was also observed following the 1995 M_W 6.9 Hanshin (Kobe) Earthquake, which experienced similar levels of ground motion as Northridge. More worrisome is that, as of this writing, some steel buildings in the San Francisco Bay Area have been found to have similar cracking, presumably as a result of the 1989 M_W 7.1 Loma Prieta Earthquake. As a result, there is an ongoing effort by a consortium of research organizations, termed SAC (funded by the Federal Emergency Management Agency) to better understand and develop solutions for this problem.

Light-gage steel buildings are used for agricultural structures, industrial factories, and warehouses. They are typically one story in height, sometimes without interior columns, and often enclose a large floor area, Figure 3.7. Construction is typically of steel frames spanning the short dimension of the building and resisting lateral forces as moment frames. Forces in the long direction are usually resisted by diagonal steel rod bracing. These buildings are usually clad with lightweight metal or asbestos reinforced concrete siding, often corrugated. Because these buildings are low rise, lightweight, and constructed of steel members, they usually perform relatively well in earthquakes. Collapses do not usually occur. Some typical problems are (a) insufficient capacity of tension braces can lead to their elongation and, in turn, building damage and (b) inadequate connection to the foundation can allow the building columns to slide.

3.4.3 Concrete Buildings

Several construction subtypes fall under this category: (a) MRFs (nonductile or ductile); (b) shear wall structures; and (c) precast, including tilt-up structures. The most prevalent of these is nonductile reinforced concrete frame structures with or without infill walls built in the United States between about 1920 and (in the western United States) 1972. In many other portions of the United States this type of construction continues to the present. This group includes large multistory commercial, institutional, and residential buildings constructed using flat slab frames, waffle slab frames, and the standard girder-column-type frames. These structures generally are more massive than steel frame buildings, are underreinforced (i.e., have insufficient reinforcing steel embedded in the concrete), and display low ductility. Some typical problems are (a) large tie spacings in columns can lead to a lack of concrete confinement and/or shear failure; (b) placement of inadequate rebar splices at the same location can lead to column failure; (c) insufficient shear strength in columns can lead to shear failure prior to the development of moment hinge capacity; (d) insufficient shear tie anchorage can prevent the column from developing its full shear capacity; (e) lack of continuous beam reinforcement can result in hinge formation during load reversal; (f) inadequate reinforcing of beam–column joints or location of beam bar splices at columns can lead to failures; and (g) the relatively low stiffness of the frame can lead to substantial nonstructural damage.

Ductile reinforced concrete frames where special reinforcing details are required in order to furnish satisfactory load-carrying performance under large deflections (termed ductility) have usually only been required in the highly seismic portions of the United States since the mid-1970s. ACI-318 (1995)

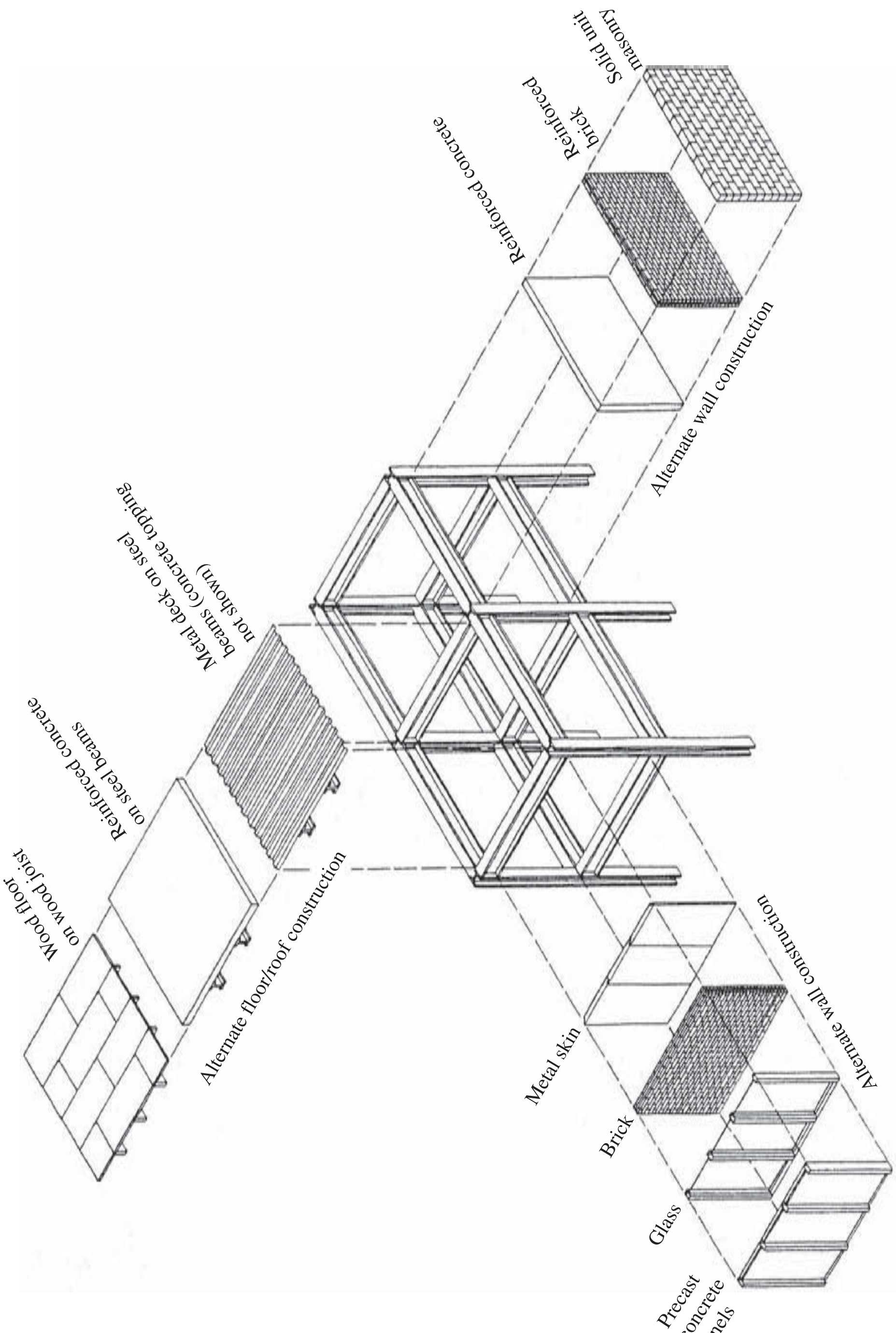


FIGURE 3.5 Steel moment resisting frame construction. (From Federal Emergency Management Agency. 1988. *Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook*, FEMA 154, FEMA, Washington, DC.)

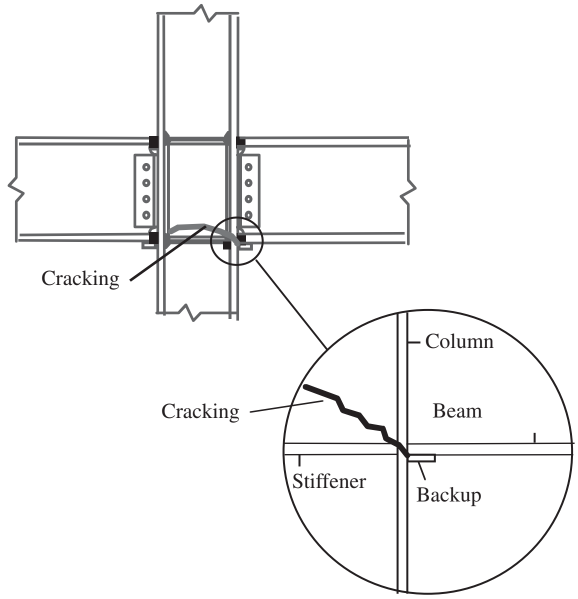


FIGURE 3.6 Example steel moment–frame connection cracking.

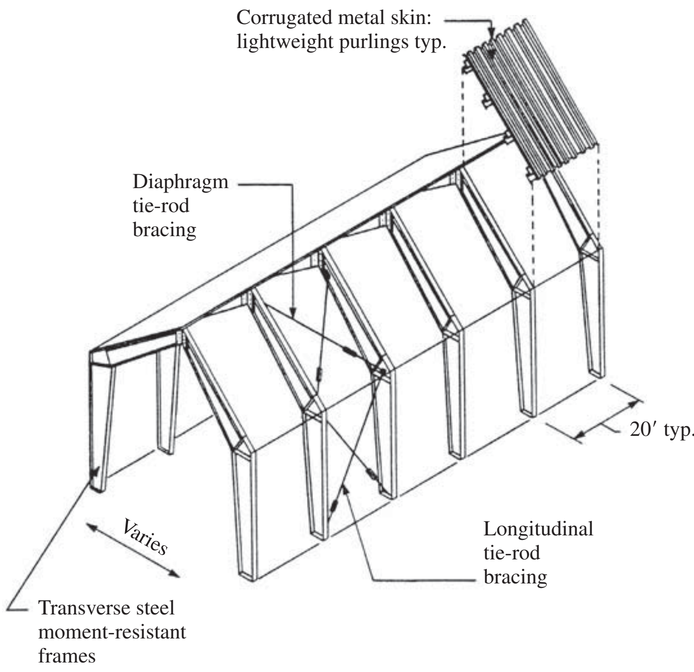


FIGURE 3.7 Light steel construction.

provides comprehensive treatment for the *ductile detailing*, which involves a number of special requirements including close spacing of lateral reinforcement in order to attain *confinement* of a concrete core, appropriate relative dimensioning of beams and columns, 135° hooks on lateral reinforcement, hooks on main beam reinforcement within the column, etc.

Concrete shear wall buildings consist of a concrete box or frame structural system with walls constituting the main LFRS, Figure 3.8. The entire structure, along with the usual concrete diaphragm, is typically cast in place. Shear walls in buildings can be located along the perimeter, as interior walls, or around the service or elevator core. This building type generally tends to perform better than concrete frame buildings. They are heavier than steel-frame buildings but they are also rigid due to the shear walls. Some types of damage commonly observed in taller buildings are caused by vertical discontinuities, pounding, and/or irregular configuration. Other damages specific to this building type are (a) shear cracking and distress can occur around openings in concrete shear walls during large seismic events; (b) shear failure can occur at wall construction joints usually at a load level below the expected capacity; and (c) bending failures can result from insufficient chord steel lap lengths.

Tilt-up buildings are a common type of construction in the western United States and consist of concrete wall panels cast on the ground and then tilted upward into their final positions. More recently, wall panels are fabricated off-site and trucked in. The wall panels are welded together at embedments or held in place by cast-in-place columns or steel columns, depending on the region. The floor and roof beams are often glue-laminated wood or steel open webbed joists that are attached to the tilt-up wall panels; these panels may be load bearing or nonload bearing, depending on the region. These buildings tend to be low-rise industrial or office buildings. Before 1973 in the western United States, many tilt-up buildings did not have sufficiently strong connections or anchors between the walls and the roof and floor diaphragms. During an earthquake, weak anchors pull out of the walls, causing the floors or roofs

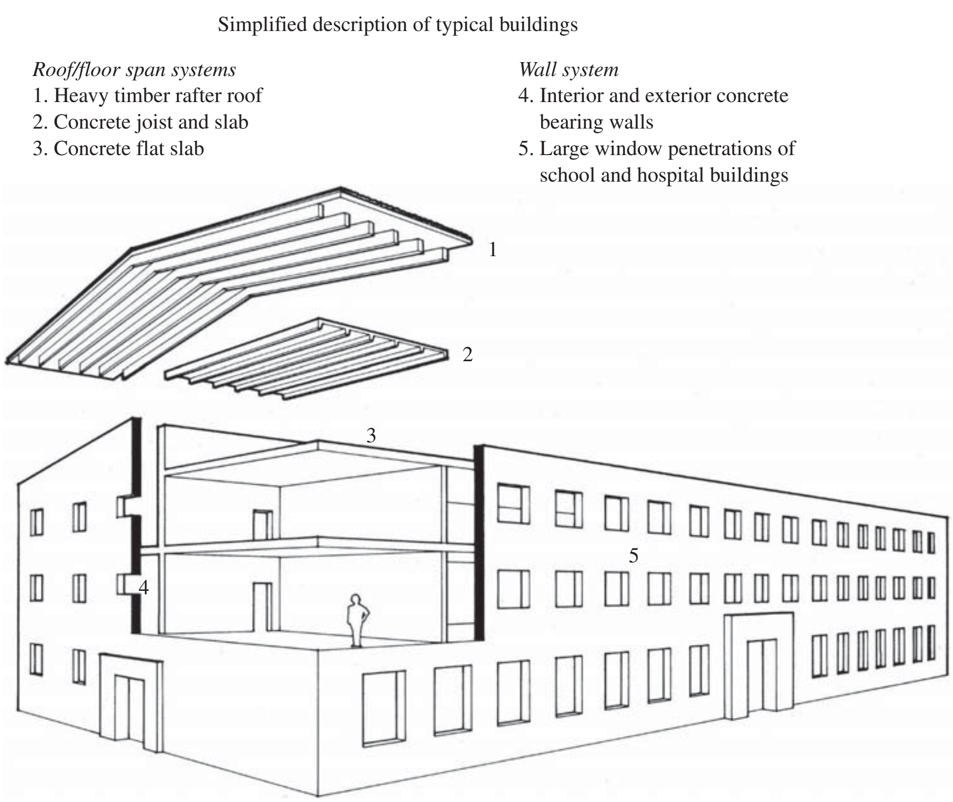


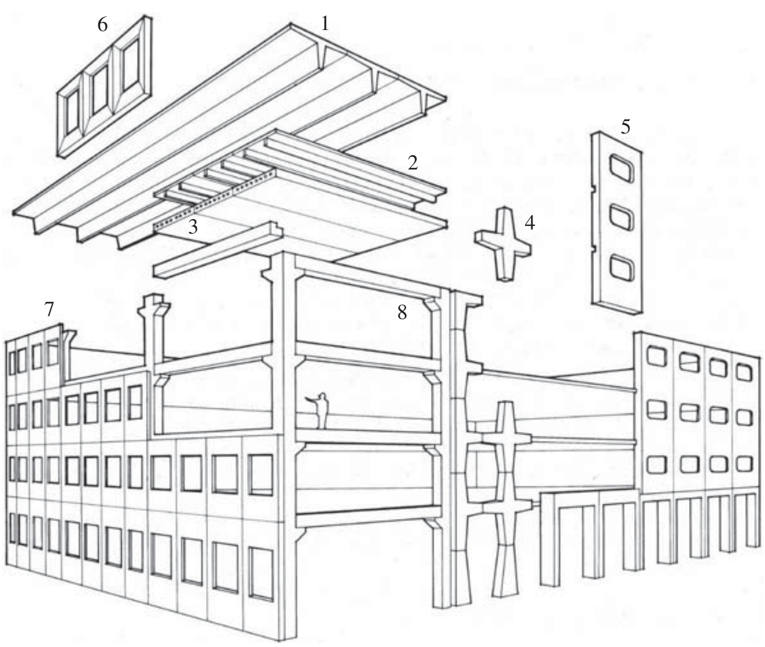
FIGURE 3.8 Reinforced concrete shear wall construction.

to collapse. The connections between concrete panels are also vulnerable to failure. Without these, the building loses much of its lateral force resisting capacity. For these reasons, many tilt-up buildings were damaged in the 1971 San Fernando Earthquake. Since 1973, tilt-up construction practices have changed in California and other high-seismicity regions, requiring positive wall–diaphragm connection and prohibiting cross-grain bending in wall ledgers. (Such requirements may not have yet been made in other regions of the country.) However, a large number of these older, pre-1970s vintage tilt-up buildings still exist and have not been retrofitted to correct this wall-anchor defect. These buildings are a prime source of seismic hazards. In areas of low or moderate seismicity, inadequate wall anchor details continue to be employed. Damage to tilt-up buildings was observed again in the 1994 M_W 6.7 Northridge earthquake, where the primary problems were poor wall anchorage into the concrete and excessive forces due to flexible roof diaphragms amplifying ground motion to a greater extent than anticipated in the code.

Precast concrete frame construction, first developed in the 1930s, was not widely used until the 1960s. The precast frame is essentially a post and beam system in concrete where columns, beams, and slabs are prefabricated and assembled on site, Figure 3.9. Various types of members are used: vertical load-carrying elements may be Ts, cross-shapes, or arches and are often more than one story in height. Beams are often Ts and double Ts or rectangular sections. Prestressing of the members, including pretensioning and posttensioning, is often employed. The LFRS is often concrete cast-in-place shear walls. The earthquake performance of this structural type varies greatly and is sometimes poor. This type of building can perform well if the details used to connect the structural elements have sufficient strength and ductility (toughness). Because structures of this type often employ cast-in-place concrete shear walls for lateral load resistance, they experience the same types of damage as other shear wall building types. Some of the problem areas specific to precast frames are (a) poorly designed connections between

Roof/floor span systems
1. Structural concrete “T” sections
2. Structural double “T” sections
3. Hollow core concrete slab

Wall systems
4. Load-bearing frame components (cross)
5. Multistory load-bearing panels



Curtain wall system
6. Precast concrete panels
7. Metal, glass, or stone panels

Structural system
8. Precast column and beams

FIGURE 3.9 Precast concrete construction.

prefabricated elements can fail; (b) accumulated stresses can result due to shrinkage and creep and due to stresses incurred in transportation; (c) loss of vertical support can occur due to inadequate bearing area and/or insufficient connection between floor elements and columns; and (d) corrosion of metal connectors between prefabricated elements can occur. A number of precast parking garages failed in the 1994 M_W 6.7 Northridge Earthquake, including a large structure at the Cal State Northridge campus that sustained a progressive failure. This structure had a perimeter precast MRF and interior nonductile columns — the MRF sustained large but tolerable deflections; however, interior nonductile columns failed under these deflections, resulting in an interior collapse, which then pulled the exterior MRFs over.

3.4.4 Masonry Buildings

Reinforced masonry buildings are mostly low-rise perimeter bearing wall structures, often with wood diaphragms although precast concrete is sometimes used. Floor and roof assemblies usually consist of timber joists and beams, glue-laminated beams, or light steel joists. The bearing walls consist of grouted and reinforced hollow or solid masonry units. Interior supports, if any, are often wood or steel columns, wood stud frames, or masonry walls. Generally, they are less than five stories in height although many mid-rise masonry buildings exist. Reinforced masonry buildings can perform well in moderate earthquakes if they are adequately reinforced and grouted and if sufficient diaphragm anchorage exists.

Most URM bearing wall structures in the western United States were built before 1934, although this construction type was permitted in some jurisdictions having moderate or high seismicity until the late 1940s or early 1950s (in low-seismicity jurisdictions URM may still be a common type of construction, even today). These buildings usually range from one to six stories in height and typically construction varies according to the type of use, although wood floor and roof diaphragms are common. Smaller commercial and residential buildings usually have light wood floor/roof joists supported on the typical perimeter URM wall and interior wood load-bearing partitions. Larger buildings, such as industrial warehouses, have heavier floors and interior columns, usually of wood. The bearing walls of these industrial buildings tend to be thick, often as much as 24 in. or more at the base. Wall thicknesses of residential buildings range from 9 in. at upper floors to 18 in. at lower floors. URM structures are recognized as perhaps the most hazardous structural type. They have been observed to fail in many modes during past earthquakes. Typical problems are

1. *Insufficient anchorage.* Because the walls, parapets, and cornices are not positively anchored to the floors, they tend to fall out. The collapse of bearing walls can lead to major building collapses. Some of these buildings have anchors as a part of the original construction or as a retrofit. These older anchors exhibit questionable performance.
2. *Excessive diaphragm deflection.* Because most of the floor diaphragms are constructed of wood sheathing, they are very flexible and permit large out-of-plane deflection at the wall transverse to the direction of the force. The large drift, occurring at the roof line, can cause the masonry wall to collapse under its own weight.
3. *Low shear resistance.* The mortar used in these older buildings is often made of lime and sand, with little or no cement, and has very little shear strength. The bearing walls will be heavily damaged and collapse under large loads.
4. *Wall slenderness.* Some of these buildings have tall story heights and thin walls. This condition, especially in nonload-bearing walls, will result in buckling out-of-plane under severe lateral load. Failure of a nonload-bearing wall represents a falling hazard, whereas the collapse of a load-bearing wall will lead to partial or total collapse of the structure.

3.4.5 Configuration, Irregularities, and Pounding

Certain problems in earthquake performance are common to many building types and include issues of configuration, irregularities, and pounding.

3.4.5.1 Configuration and Irregularities

Configuration, or the general vertical and/or horizontal shape of buildings, is an important factor in earthquake performance and damage. Buildings that have simple, regular, symmetric configurations generally display the best performance in earthquakes. The reasons for this are (a) nonsymmetric buildings tend to have twist (i.e., have significant torsional modes) in addition to shaking laterally and (b) the various "wings" of a building tend to act independently, resulting in differential movements, cracking, and other damage. Rotational motion introduces additional damage, especially at re-entrant or "internal" corners of the building. The term "configuration" also refers to the geometry of lateral load resisting systems as well as the geometry of the building. Asymmetry can exist in the placement of bracing systems, shear walls, or MRFs that are used to provide earthquake resistance in a building. This type of asymmetry, of the LFRS, can result in twisting or differential motion, with the same consequences as asymmetry in the building plan. An important aspect of configuration is *soft story*, which is a story of a building signifiantly less stiff than adjacent stories (i.e., a story in which the lateral stiffness is 70% or less than that in the story above or less than 80% of the average stiffness of the three stories above; BSSC 2001). Soft stories often (but not always) occur on the ground floor, where commercial or other reasons require a greater story height, and large windows or openings for ingress or commercial display (e.g., the building might have masonry curtain walls for the full height, except at the ground floor, where these are replaced with large windows for a store's display). Due to inadequate stiffness, a disproportionate amount of the entire building's drift is concentrated at the soft story, resulting in nonstructural and potential structural damage. Many older buildings with soft stories but built prior to recognition of this aspect collapse due to excessive ductility demands at the soft story.

The National Earthquake Hazard Reduction Program (NEHRP) provisions for the design of new buildings (BSSC 2001) have defined when a building's configuration is "irregular," and provided required strength increase factors or other approaches to deal with these irregularities. The NEHRP definitions for plan and vertical irregularities are illustrated in Table 3.1 and Table 3.2.

3.4.5.2 Pounding

Pounding is the collision of adjacent buildings during an earthquake due to insufficient lateral clearance. Such collision can induce very high and unforeseen accelerations and story shears in the overall structure. Additionally, if adjacent buildings have varying story heights, a relatively rigid floor or roof diaphragm may impact adjacent buildings at or near mid-column height, causing bending or shear failure in the columns and subsequent story collapse. Under earthquake lateral loading, buildings deflect significantly — these deflections or drift are limited by code — and adjacent buildings must be separated by a seismic gap equal to the sum of their actual calculated drifts (i.e., ideally, each building set back from its property line by the drift). Pounding has been the cause of a number of mid-rise building collapses, most notably in the 1985 Mexico City Earthquake.

3.5 2000 NEHRP Recommended Provisions

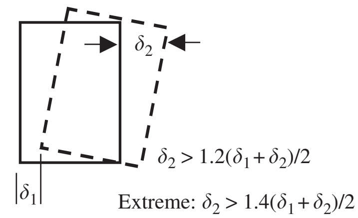
3.5.1 Overview

The *2000 NEHRP Recommended Provisions for Seismic Regulation for Buildings and Other Structures* (NEHRP Provisions) represents the current state of the art in prescriptive, as opposed to performance-based, provisions for seismic-resistant design. Its provisions form the basis for earthquake design specifications contained in the 2001 edition of *ASCE-7, Minimum Design Loads for Buildings and other Structures*, either through reference or direct incorporation, the seismic regulations in the 2003 edition of the *International Building Code*, and also the 2002 edition of the *NFPA 5000 Building Code* (NFPA, n.d.). As such, it will form the basis for most earthquake-resistant design in the United States, as well as other nations that base their codes on U.S. practices, throughout much of the first decade of the twenty-first century.

TABLE 3.1 NEHRP 2000 Plan Structural Irregularities

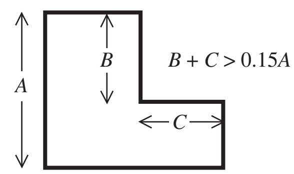
1: Torsional irregularity — when diaphragms are not flexible

Torisonal irregularity exists (1a) when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drift at the two ends of the structure. (1b) Extreme torisonal irregularity exists when ratio >1.4



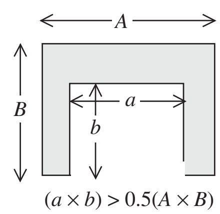
2: Re-entrant corners

Plan configurations of a structure and its lateral-force-resisting system contain re-entrant corners where both projections of the structure beyond a re-entrant corner are greater than 15% of the plan dimension of the structure in the given direction



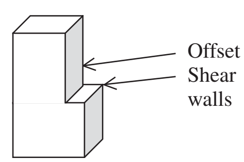
3: Diaphragam discontinuity

Diaphragms with abrupt discontinuities or variations in stiffness including those having cutout or open areas greater than 50% of the gross enclosed diaphragm area or changes in effective diaphragm stiffness of more than 50% from one story to the next



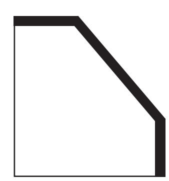
4: Out-of-plane offsets

Discontinuities in a lateral-force-resistance path such as out-of-plane offsets of the vertical elements



5: Nonparallel systems

The vertical lateral-force-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force-resisting system



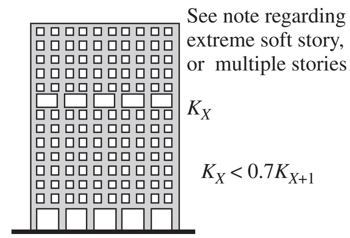
The *NEHRP Provisions* assume significant amounts of nonlinear behavior will occur under design level events. The extent of nonlinear behavior that may occur is dependent on the structural systems employed in resisting earthquake forces, the configuration of these systems, and the extent that the structural systems are detailed for ductile behavior under large cyclic inelastic deformation. The *NEHRP Provisions* may therefore be thought to consist of two component parts:

- One part relates to specification of the required design strength and stiffness of the structural system.
- The second part relates to issues of structural detailing.

TABLE 3.2 NEHRP 2000 Vertical Structural Irregularities

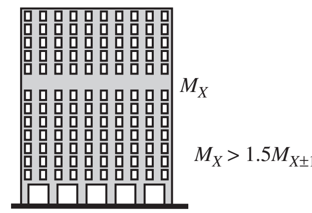
1: Stiffness irregularity — soft story

(1a) A soft *story* is one in which the lateral stiffness is less than 70% of that in the *story* above or less than 80% of the average stiffness of the three stories above. (1b) An extreme soft *story* is for ratios of less than 60% or less than 70%, respectively



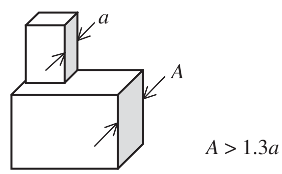
2: Weight (mass) irregularity

Mass irregularity exists where the effective mass of any *story* is more than 150% of the effective mass of an adjacent *story*. A roof that is lighter than the floor below need not be considered



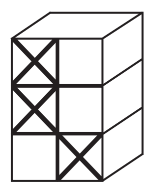
3: Vertical geometric irregularity

Vertical geometric irregularity exists where the horizontal dimension of the lateral-force-resisting system in any *story* is more than 130% of that in an adjacent *story*



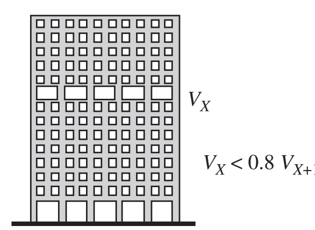
4: In-plane discontinuity in vertical lateral-force-resisting elements

An in-plane offset of the lateral-force-resisting elements greater than the length of those elements or a reduction in stiffness of the resisting elements in the story below



5: Discontinuity in capacity — weak story

A weak *story* is one in which the *story* lateral *strength* is less than 80% of that in the *story* above. The *story strength* is the total *strength* of all seismic-resisting elements sharing the *story* shear for the direction under consideration



For this second part, the *NEHRP Provisions* adopt, with modification, design standards and specifications developed by industry groups such as the American Concrete Institute or the American Institute of Steel Construction. This second part of the *NEHRP Provisions* is not discussed in this chapter, but is covered in detail in each of the following chapters, which treat the individual structural materials.

Instead, this article focuses primarily on the manner in which the *NEHRP Provisions* regulate the required strength and stiffness of structures.

3.5.2 Performance Intent and Objectives

The *NEHRP Provisions* are intended to provide a tiered series of performance capabilities for structures, depending on their intended occupancy and use. Under the *NEHRP Provisions*, each structure must be assigned to a seismic use group (SUG). Three SUGs are defined and are respectively labeled I, II, and III:

- SUG-I encompasses most ordinary occupancy buildings including typical commercial, residential, and industrial structures. For these facilities the basic intent of the *NEHRP Provisions*, just as with earlier codes, is to provide a low probability of earthquake-induced life safety endangerment.
- SUG-II includes facilities that house large numbers of persons, persons who are mobility impaired, or large quantities of materials that if released could pose substantial hazards to the surrounding community. Examples of such facilities include large assembly facilities, housing several thousand persons, daycare centers, and manufacturing facilities containing large quantities of toxic or explosive materials. The performance intent for these facilities is to provide a lower probability of life endangerment, relative to SUG-I structures, and a low probability of damage that would result in release of stored materials.
- SUG-III includes those facilities such as hospitals and emergency operations and communications centers deemed essential to disaster response and recovery operations. The basic performance intent of the *NEHRP Provisions* with regard to these structures is to provide a low probability of earthquake-induced loss of functionality and operability.

In reality, the probability of damage resulting in life endangerment, release of hazardous materials, or loss of function should be calculated using structural reliability methods as the total probability of such damage over a period of time (Ravindra 1994). Mathematically, this is equal to the integral, over all possible levels of ground motion intensity, of the conditional probability of excessive damage given that a ground motion intensity is experienced and the probability that such ground motion intensity will be experienced in the desired period of time. Although such an approach would be mathematically and conceptually correct, it is currently regarded as too complex for practical application in the design office.

Instead, the *NEHRP Provisions* design for desired limiting levels of nonlinear behavior for a single design earthquake intensity level, termed *maximum considered earthquake* (MCE) ground shaking. In most regions of the United States, the MCE is defined as that intensity of ground shaking having a 2% probability of exceedance in 50 years. In certain regions, proximate to major active faults, this probabilistic definition of MCE motion is limited by a conservative deterministic estimate of the ground motion intensity anticipated to result from an earthquake of characteristic magnitude on these faults. The MCE is thought to represent the most severe level of shaking ever likely to be experienced by a structure, though it is recognized that there is some limited possibility of more severe motion occurring.

Structures categorized as SUG-I are designed with the expectation that MCE shaking would result in severe damage to both structural and nonstructural elements, with damage perhaps being so severe that following the earthquake the structure would be on the verge of collapse. This damage state has come to be termed *collapse prevention*, because the structure is thought to be at a state of incipient but not actual collapse. Theoretically, SUG-I structures behaving in this manner would be total or near total financial losses, in the event that MCE shaking was experienced. To the extent that shaking experienced by the structure exceeds the MCE level, the structure could actually experience partial or total collapse.

SUG-III structures are designed with the intent that when subjected to MCE shaking they would experience both structural and nonstructural damages; however, the structures would retain significant residual structural resistance or margin against collapse. It is anticipated that when experiencing MCE shaking such structures may be damaged to an extent that they would no longer be suitable for occupancy, until repair work had been instituted, but that repair would be technically and economically feasible. This superior performance relative to SUG-I structures is accomplished through specification that SUG-III structures be designed with 50% greater strength and more stiffness than their SUG-I counterparts. SUG-II structures are designed for performance intermediate to that for SUG-I and SUG-III with strengths and stiffness that are 25% greater than those required for SUG-I structures.

3.5.3 Seismic Hazard Maps and Ground Motion Parameters

The *NEHRP Provisions* incorporate a series of national seismic hazard maps for the United States and territories, developed by the United States Geologic Survey (USGS), specifically for this purpose (available at http://geohazards.cr.usfs.gov/eq/index.html). Two sets of maps are presented. One set presents contours of MCE, 5% damped, elastic spectral response acceleration at a period of 0.2 s, termed S_S. The second set presents contours of MCE, 5% damped, elastic spectral response acceleration at a period of 1.0 s, termed S_1. In both cases, the spectral response acceleration values are representative of sites with subsurface conditions bordering between firm soil or soft rock. Contours are presented in increments of 0.02*g* in areas of low seismicity and 0.05*g* in areas of high seismicity. By locating a site on the maps and interpolating between the values presented for contours adjacent to the site, it is possible to rapidly estimate the MCE level shaking parameters for the site, given that it has a soft rock or firm soil profile Figure 3.10 shows, for a portion of the western United States, contours of the 0.2 s spectral acceleration with a 90% probability of not being exceeded in 50 years. As indicated in the figure, in zones of high seismicity these contours are quite closely spaced, making use of the maps difficult. Therefore, the USGS has furnished software, available both over the internet (at the URL indicated above) and on a CD-ROM, which permits determination of the MCE spectral response acceleration parameters based on longitude and latitude.

Since many sites are located neither on soft rock nor on firm soil sites, it is necessary to correct the mapped values of spectral response acceleration to account for site amplification and de-amplification effects. To facilitate this process, a site is categorized into one of six site class groups, labeled A through F. Table 3.3 summarizes the various site class categories.

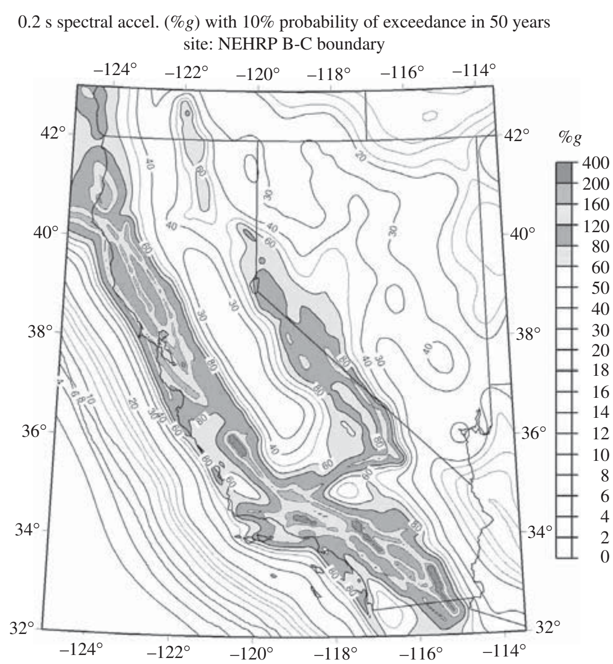


FIGURE 3.10 MCE Seismic Hazard Map (0.2 s spectral response acceleration) for western United States.

TABLE 3.3 Site Categories

Site class	Description	Shear wave velocity, $\bar{v}_s$	Penetration resistance, $\bar{N}$	Unconfined shear strength, $\bar{s}_u$
A	Hard rock	>5000 ft/s		
B	Rock	2500 ft/s $< \bar{v}_s \leq$ 5000 ft/s		
C	Very firm soil or soft rock	1200 ft/s $< \bar{v}_s \leq$ 2500 ft/s	>50	>2000 psf
D	Stiff soil	600 ft/s $< \bar{v}_s \leq$ 1200 ft/s	15–50	1000–2000 psf
E	Soil	$\bar{v}_s <$ 600 ft/s	<15	<1000 psf
F	Special soils	Soils requiring site-specific evaluations: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils 2. Peats and/or highly organic clays ($H >$ 10 ft of peat and/or highly organic clay where $H =$ thickness of soil) 3. Very high plasticity clays ($H >$ 25 ft [8 m] with PI $>$ 75) 4. Very thick soft/medium stiff clays ($H >$ 120 ft [36 m])		

Note: $\bar{v}_s$, $\bar{N}$, $\bar{s}_u$ represent the average value of the parameter over the top 30 m (100 ft) of soil.

TABLE 3.4 Coefficient F_a as a Function of Site Class and Mapped Spectral Response Acceleration

	Mapped maximum considered earthquake spectral response acceleration at short periods				
Site class	$S_S = 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S = 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	a
F	a	a	a	a	a

Note: "a" indicates site-specific evaluation required.

Once a site has been categorized within a site class, a series of coefficients are provided that are used to adjust the mapped values of spectral response acceleration for site response effects. These coefficients were developed based on observed site response characteristics in ground motion recordings from past earthquakes. Two coefficients are provided:

- The F_a coefficient is used to account for site response effects on short period ground shaking intensity.
- The F_v coefficient is used to account for site response effects of longer period motions.

Table 3.4 and Table 3.5 indicate the values of these coefficients as a function of site class and mapped MCE ground shaking acceleration values. Site-adjusted values of the MCE spectral response acceleration parameters at 0.2 and 1 s, respectively, are found from the following equations:

$$S_{MS} = F_a S_s \tag{3.1}$$

$$S_{M1} = F_v S_1 \tag{3.2}$$

The two site-adjusted spectral response acceleration parameters, S_{MS} and S_{M1}, permit a 5% damped, maximum considered earthquake ground shaking response spectrum to be constructed for the building site. This spectrum is constructed as indicated in Figure 3.11 and consists of a constant response acceleration range, between periods of T_0 and T_s, a constant response velocity range for periods in excess of T_s, and a short period range that ramps between an estimated zero period acceleration given by $S_{MS}/2.5$ and S_{MS}. Site-specific spectra can also be used. Regardless of whether site-specific spectra or

TABLE 3.5 Coefficient F_v as a Function of Site Class and Mapped Spectral Response Acceleration

	Mapped maximum considered earthquake spectral response acceleration at 1 s periods				
Site class	$S_1 = 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	a
F	a	a	a	a	a

Note: "a" indicates site-specific evaluation required.

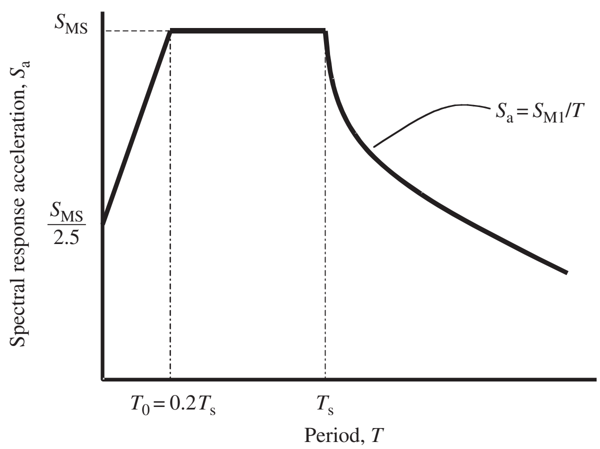


FIGURE 3.11 Maximum considered earthquake response spectrum.

spectra based on mapped values are used, the actual design values are taken as two thirds of the MCE values. The resulting design parameters are, respectively, labeled S_{DS} and S_{D1} and the design spectrum is identical to the MCE spectrum, except that the ordinates are taken as two thirds of the MCE values. The reason for using design values that are two thirds of the maximum considered values is that the design procedures, described in later sections, are believed to provide a minimum margin against collapse of 150%. Therefore, if design is conducted for two thirds of the MCE ground shaking, it is anticipated that buildings experiencing MCE ground shaking would be at incipient collapse, the desired performance objective for SUG-I structures.

3.5.4 Seismic Design Categories

The seismicity of the United States, and indeed the world, varies widely. It encompasses zones of very high seismicity in which highly destructive levels of ground shaking are anticipated to occur every 50 to 100 years and zones of much lower seismicity in which only moderate levels of ground shaking are ever anticipated. The *NEHRP Provisions* recognize that it is neither technically necessary nor economically appropriate to require the same levels of seismic protection for all buildings across these various regions of seismicity. Instead, the *NEHRP Provisions* assign each structure to a seismic design category (SDC) based on the level of seismicity at the building site, as represented by mapped shaking parameters, and the SUG.

Six SDCs, labeled A through F, are defined. SDC A represents the least severe seismic design condition and includes structures of ordinary occupancy located on sites anticipated to experience only very limited levels of ground shaking. SDC F represents the most severe design condition and includes

TABLE 3.6 Categorization of Structures into Seismic Design Category, Based on Design Short Period Spectral Response Acceleration, S_{DS} and Seismic Use Group

	Seismic Use Group		
Value of S_{DS}	I	II	III
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D[a]	D[a]	D[a]

[a] *Seismic Use Group* I and II *structures* located on sites with mapped *maximum considered earthquake* spectral response acceleration at 1 s period, S_1, equal to or greater than $0.75g$ shall be assigned to *Seismic Design Category* E and *Seismic Use Group* III *structures* located on such sites shall be assigned to *Seismic Design Category* F.

TABLE 3.7 Categorization of Structures into Seismic Design Category, Based on Design One-Second Period Spectral Response Acceleration, S_{D1} and Seismic Use Group

	Seismic Use Group		
Value of S_{D1}	I	II	III
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D[a]	D[a]	D[a]

[a] See footnote to Table 3.6.

structures assigned to SUG-III and located within a few kilometers of major, active faults, anticipated to produce very intense ground shaking. A designer determines to which SDC a structure should be assigned by reference to a pair of tables, reproduced as Table 3.6 and Table 3.7. A structure is assigned to the most severe category indicated by either table.

Nearly all aspects of the seismic design process are affected by the SDC that a structure is assigned to. This includes designation of the permissible structural systems, specification of required detailing, limitation on permissible heights and configuration, the types of analyses that may be used to determine the required lateral strength and stiffness, and the requirements for bracing and anchorage of non-structural components.

3.5.5 Permissible Structural Systems

The *NEHRP Provisions* define more than 70 individual seismic-force-resisting system types. These systems may be broadly categorized into five basic groups that include: bearing wall systems, building frame systems, moment-resisting frame systems, dual systems, and special systems:

- Bearing wall systems include those structures in which the vertical elements of the LFRS comprise either shear walls or braced frames in which the shear-resisting elements (walls or braces) are required to provide support for gravity (dead and live) loads in addition to providing lateral resistance. This is similar to the “box system” contained in earlier codes.
- Building frame systems include those structures in which the vertical elements of the LFRS comprise shear walls or braces, but in which the shear-resisting elements are not also required to provide support for gravity loads.
- Moment-resisting frame systems are those structures in which the lateral-force resistance is provided by the flexural rigidity and strength of beams and columns, which are interconnected in such a manner that stress is induced in the frame by lateral displacements.

- Dual systems rely on a combination of MRFs and either braced frames or shear walls. In dual systems, the braced frames or shear walls provide the primary lateral resistance and the MRF is provided as a back-up or redundant system, to provide supplemental lateral resistance in the event that earthquake response damages the primary lateral-force-resisting elements to an extent that they lose effectiveness.
- Special systems include unique structures, such as those that rely on the rigidity of cantilevered columns for their lateral resistance.

Within these broad categories, structural systems are further classified in accordance with the quality of detailing provided and the resulting ability of the structure to withstand earthquake-induced inelastic, cyclic demands. Structures that are provided with detailing believed capable of withstanding large cyclic inelastic demands are typically termed “special” systems. Structures that are provided with relatively little detailing and therefore, incapable of withstanding significant inelastic demands are termed “ordinary.” Structures with limited levels of detailing and inelastic response capabilities are termed “intermediate.” Thus, within a type of structure, for example, moment-resisting steel frames or reinforced concrete bearing walls, it is possible to have “special” MRFs or bearing walls, “intermediate” MRFs or shear walls, and “ordinary” MRFs or shear walls. The various combinations of such systems and construction materials results in a wide selection of structural systems to choose from. The use of “ordinary” and “intermediate” systems, regarded as having limited capacity to withstand cyclic inelastic demands, is generally limited to SDC A, B, and C and to certain low-rise structures in SDC D.

3.5.6 Design Coefficients

Under the *NEHRP Provisions*, required seismic design forces and, therefore, required lateral strength is typically determined by elastic methods of analysis, based on the elastic dynamic response of structures to design ground shaking. However, since most structures are anticipated to exhibit inelastic behavior when responding to the design ground motions, it is recognized that linear response analysis does not provide an accurate portrayal of the actual earthquake demands. Therefore, when linear analysis methods are employed, a series of design coefficients are used to adjust the computed elastic response values to suitable design values that consider probable inelastic response modification. Specifically, these coefficients are the response modification factor, R, the overstrength factor, Ω_0, and the deflection amplification coefficient, C_d. Tabulated values of these factors are assigned to a structure based on the selected structural system and the level of detailing employed in that structural system:

- The response modification coefficient, R, is used to reduce the required lateral strength of a structure, from that which would be required to resist the design ground motion in a linear manner to that required to limit inelastic behavior to acceptable levels, considering the characteristics of the selected structural system. Structural systems deemed capable of withstanding extensive inelastic behavior are assigned relatively high R values, as large as eight, permitting minimum design strengths that are only $\frac{1}{8}$ that required for elastic response to the design motion. Systems deemed to be incapable of providing reliable inelastic behavior are assigned low R values, approaching unity, requiring sufficient strength to resist design motion in a nearly elastic manner.
- The deflection amplification coefficient, C_d, is used to estimate the total elastic and inelastic lateral deformations of the structure when subjected to design earthquake ground motion. Specifically, lateral deflections calculated for elastic response of the structure to the design ground motion, reduced by the response modification coefficient R, are amplified by the factor C_d to obtain this estimate. The C_d coefficient accounts for the effects of viscous and hysteretic damping on structural response, as well as the effects of inelastic period lengthening. Structural systems that are deemed capable of developing significant amounts of viscous and hysteretic damping are assigned C_d values somewhat less than the value of the R coefficient. This results in an estimate of total lateral deformation that is somewhat lower than would be anticipated for a pure elastic response.

For structural systems with relatively poor capability to develop viscous and/or hysteretic damping, the C_d value may exceed R, resulting in estimates of lateral drift that exceed that calculated for elastic response.

- The overstrength coefficient, Ω_0, is used to provide an estimate of the maximum force likely to be delivered to an element in the structure, considering that due to effects of system and material overstrength this may be larger than the force calculated by elastic analysis of the structure's response to design ground motion, reduced by the response modification coefficient R. This overstrength factor is used to compute the required strength to resist behavioral modes that have limited capacity for inelastic response, such as column buckling or connection failure in braced frames.

Figure 3.12 illustrates the basic concepts behind these design coefficients. The figure contains an elastic design response spectrum, an elastic response line, and an inelastic response curve for an arbitrary structure, all plotted in lateral inertial force (base shear) versus lateral roof displacement coordinates. Response spectra are more familiarly plotted in coordinates of spectral response acceleration (S_a) versus structural period (T). It is possible to convert a spectrum plotted in that form to the spectrum shown in the figure through a two-step process. The first step consists of converting the response spectrum for S_a versus T coordinates to S_a versus spectral response displacement (S_d) coordinates. This is performed using the following relationship between S_a, S_d, and T:

$$S_d = \frac{T^2}{4\pi^2} S_a \tag{3.3}$$

Then the response spectrum is converted to the form shown in the figure by recognizing that for a structure responding in a given mode of excitation the base shear is equal to the product of the mass participation factor for that mode, the structure's mass, and the spectral response acceleration, S_a, at that period. Similarly, the lateral roof displacement for a structure responding in that mode is equal to the spectral response displacement times the modal participation factor. For a single degree of freedom structure, the mass participation factor and modal participation factor are both unity and the lateral base shear, V, is equal to the product of the spectral response acceleration at the mode of response and the mass of the structure, while the lateral roof displacement is equal to the spectral response displacement.

The dashed diagonal line in the figure represents the elastic response of the arbitrary structure. It is a straight line because a structure responding in an elastic manner will have constant stiffness and, therefore, a constant proportional relationship between the applied lateral force and resulting

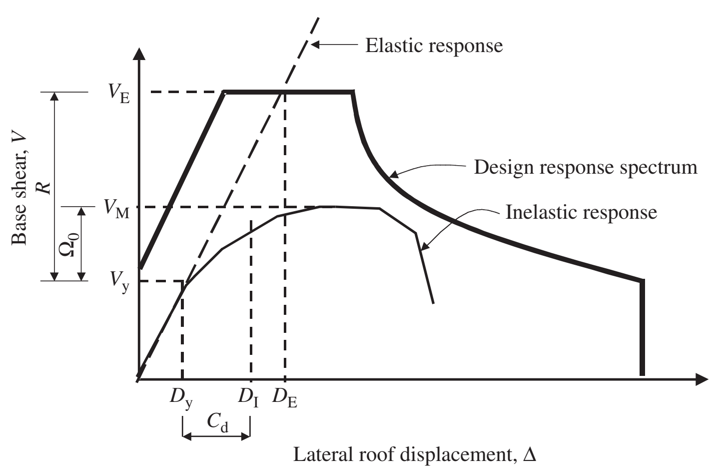


Figure 3.12 Schematic illustration of design coefficients.

displacement. The intersection of this diagonal line with the design response spectrum indicates the maximum total lateral base shear, V_E, and roof displacement, D_E, of the structure would develop if it responded to the design ground motion in an elastic manner. The third plot in the figure represents the inelastic response characteristics of this arbitrary structure, sometimes called a pushover curve. The pushover curve has an initial elastic region having the same stiffness as the elastic response line. The point V_y, D_y on the pushover curve represents the end of this region of elastic behavior. Beyond V_y, D_y, the curve is represented by a series of segments, with sequentially reduced stiffness, representing the effects of inelastic softening of the structure. The lateral base shear force, V_M, at the peak of the pushover curve, represents the maximum lateral force that the structure is capable of developing at full yield.

The response modification coefficient, R, is used in the provisions to set the minimum acceptable strength at which the structure will develop its first significant yielding, V_y. This is given by the simple relationship

$$V_y = \frac{V_E}{R} \tag{3.4}$$

The coefficient Ω_0 is used to approximate the full yield strength of the structure through the relationship

$$V_M = \Omega_0 V_y \tag{3.5}$$

The maximum total drift of the structure, D_I, is obtained from the relationship

$$D_I = C_d D_y \tag{3.6}$$

3.5.7 Analysis Procedures

The *NEHRP Provisions* permit the use of five different analytical procedures to determine the required lateral strength of a structure and to confirm that the structure has adequate stiffness to control lateral drift. The procedures permitted for a specific structure are dependent on the structure's SDC and its regularity.

3.5.7.1 Index Force Procedure

The index force procedure is permitted only for structures in SDC A. In this procedure, the structure must be designed to have sufficient strength to resist a static lateral force equal to 1% of the weight of the structure, applied simultaneously to each level. The forces must be applied independently in two orthogonal directions. Structures in SDC A are not anticipated ever to experience ground shaking of sufficient intensity to cause structural damage, provided that the structures are adequately tied together and have a complete LFRS. The nominal, 1%, lateral force function used in this procedure is intended as a means of ensuring that the structure has a complete LFRS of nominal, though somewhat arbitrary, strength. In addition to providing protection for the low levels of ground motion, anticipated for SDC A structures, this procedure is also considered to be a structural integrity provision, intended to provide nominal resistance against blast and other possible loading events.

3.5.7.2 Equivalent Lateral Force Analysis

The estimation of forces an earthquake may impose on a building can be accomplished by use of an equivalent lateral force (ELF) procedure. The ELF is commonly employed for simpler buildings and is provided in most building codes. The reader is referred to Hamburger (2002) for a review of building code and ELF development, but it will be noted that for many years the uniform building code (UBC) and other building codes determined the ELF according to variants on the equation $V = ZICKSW$, where V was the ELF or total design lateral force applied to the structure (or shear at the structure's base), determined from Z, a seismic zone factor varying between 0.075 (Zone 1, low-seismicity areas) and 0.40 (Zone 4, high-seismicity areas); I, an importance factor varying between 1.0 and 1.5; C, a function of the structure's fundamental period T, which effectively defined a response spectral shape; K, a factor varying by type of structure that accounted for the ductility of the structure and its material; S, a factor to account for site soil conditions; and W, the total seismic dead load.

In the 2000 *NEHRP Provisions*, ELF analysis may be used for any structure in SDC B and C, for any structure of light-frame construction, and for all regular structures, with a calculated structural period, T, not greater than $3.5T_s$, where T_s is as previously defined in Figure 3.11. ELF analysis consists of a simple approximation to modal response spectrum analysis. It only considers the first mode of a structure's lateral response and presumes that the mode shape for this first mode of response is represented by that of a simple shear beam. For structures having sufficiently low periods of first mode response ($T < 3.5T_s$) and regular vertical and horizontal distribution of stiffness and mass, this procedure approximates modal response spectrum analysis well. However, for longer period structures, higher mode response becomes significant and neglecting these higher modes results in significant errors in the estimation of structural response. Also, as the distribution of mass and stiffness in a structure becomes irregular, for example, the presence of torsional conditions or soft story conditions, the assumptions inherent in the procedure with regard to mode shape also become quite approximate, leading to errors. In SDCs D, E, and F, this method is permitted only for those structures where these inaccuracies are unlikely to be significant. The procedure is permitted for more general use in other SDCs because it is felt that the severity of design ground motion is low enough that inaccuracies in analysis of lateral response is unlikely to result in unacceptable structural performance and also because it is felt that designers in these regions of low seismicity may not be able to implement the more sophisticated and accurate methods properly.

As with the index force analysis procedure, the ELF consists of the simultaneous application of a series of static lateral forces to each level of the structure in each of the two independent orthogonal directions. In each direction, the total lateral force, known as the base shear, is given by the formula

$$V = \frac{S_{DS}}{R/I} W \tag{3.7}$$

This formula gives the maximum lateral inertial force that acts on an elastic, single degree of freedom structure with a period that falls within the constant response acceleration (periods shorter than T_s) portion of the design spectrum, reduced by the term R/I. In this formula, S_{DS} is the design spectral response acceleration at short periods, W is the dead weight of the structure and a portion of the supported live load, R is the response modification coefficient, and I is an occupancy importance factor, assigned based on the structure's SUG. For SUG-1 structures, I is assigned a value of unity. For SUG II and III structures, I is assigned values of 1.25 and 1.5, respectively. The effect of I is to reduce the permissible response modification factor, R, for structures in higher SUGs, requiring that the structures have greater strength, thereby limiting the permissible inelasticity and damage in these structures.

The base shear force given by Equation 3.7 need never exceed the following:

$$V = \frac{S_{D1}}{(R/I)T} W \tag{3.8}$$

Equation 3.8 represents the maximum lateral inertial force that acts on an elastic, single degree of freedom structure with period T that falls within the constant response velocity portion of the design spectrum (periods longer than T_s), reduced by the response modification coefficient, R and the occupancy importance factor, I. In this equation, all terms are as previously defined except that S_{D1} is the design spectral response acceleration at 1 s. For short period structures, Equation 3.7 will control. For structures with periods in excess of T_s, Equation 3.8 will control.

The shape of the design response spectrum shown in Figure 3.11 is not representative of the dynamic characteristics of ground motion found close to the fault rupture zone. Such motions are often dominated by a large velocity pulse and very large spectral displacement demands. Therefore, for structures in SDCs E and F, the seismic design categories for structures located close to major active faults, the base shear may not be taken less than the value given by Equation 3.9. Equation 3.9 approximates the effects of the additional long period displacements that have been recorded in some near field ground motion records

$$V = \frac{0.5S_1}{R/I} \tag{3.9}$$

The total, lateral base shear force given by Equations 3.7–3.9 must be distributed vertically for application to the various mass or diaphragm levels of the structure. For a structure with n levels, the force at diaphragm level x is given by the equation

$$F_x = C_{vx}V \tag{3.10}$$

where

$$C_{vx} = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{3.11}$$

h_x and h_i, respectively, are the heights of levels x and i above the structure's base. These formula are based on the assumption that the structure is responding in its first mode, in pure sinusoidal motion, and that the mode shape is linear. That is, it is assumed that at any instant of time, the displacement at level x of the structure is

$$\delta_x = \frac{h_x}{h_n}\delta_n \tag{3.12}$$

where δ_x and δ_n are the lateral displacements at level x and the roof of the structure, respectively, and h_n is the total height of the structure. For a structure responding in pure sinusoidal motion, the displacement δ_x, velocity v_x, and acceleration a_x, of level x at any instant of time, t, is given by the following equations:

$$\delta_x = \delta_{x\,\max} \sin\left(\frac{2\pi}{T}t\right) \tag{3.13}$$

$$v_x = \delta_{x\,\max} \frac{2\pi}{T} \cos\left(\frac{2\pi}{T}t\right) \tag{3.14}$$

$$a_x = -\delta_{x\,\max} \frac{4\pi}{T^2} \sin\left(\frac{2\pi}{T}t\right) \tag{3.15}$$

Since acceleration at level x is directly proportional to the displacement at level x, the acceleration at level x in a structure responding in pure sinusoidal motion is given by the equation

$$a_x = \frac{h_n}{h_x}a_n \tag{3.16}$$

where a_n is the acceleration at the roof level. Since the inertial force at level x is equal to the product of mass at level x and the acceleration at level x, Equation 3.11 can be seen to be an accurate distribution of lateral inertial forces in a structure responding in a linear mode shape.

The lateral forces given by Equation 3.10 are applied to a structural model of the building and the resulting member forces and building interstory drifts are determined. The analysis must consider the relative rigidity of both the horizontal and vertical elements of the LFRS, and when torsional effects are significant, must consider three-dimensional distributions of stiffness, centers of mass, and rigidity. The structure must then satisfy two basic criteria. First, the elements of the LFRS must have sufficient strength to resist the calculated member forces in combination with other loads, and second, the structure must have sufficient strength to maintain computed interstory drifts within acceptable levels. The specific load combinations that must be used to evaluate member strength and the permissible interstory drifts are described in succeeding sections.

In recognition of the fact that higher mode participation can result in significantly larger forces at individual diaphragm levels, than is predicted by Equation 3.11, forces on diaphragms are computed using an alternative equation, as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \tag{3.17}$$

where F_{px} is the design force applied to diaphragm level x, F_i is the force computed from Equation 3.11 at level i, w_{px} is the effective seismic weight, at level x, and w_i is the effective weight at level i.

3.5.7.3 Response Spectrum Analysis

Response spectrum analysis is permitted to be used for the design of any structure. The procedure contained in the *NEHRP Provisions* uses standard methods of elastic modal dynamic analysis, which are not described here, but are well documented in the literature, for example, by Chopra (1981). The analysis must include sufficient modes of vibration to capture participation of at least 90% of the structure's mass in each of the two orthogonal directions. The response spectrum used to characterize the loading on the structure may be either the generalized design spectrum for the site, shown in Figure 3.11, or a site-specific spectrum developed considering the regional seismic sources and site characteristics.

Regardless of the spectrum used, the ground motion is scaled by the factor (I/R), just as in the ELF technique. The *NEHRP Provisions* require that the member forces determined by response spectrum analysis be scaled so that the total applied lateral force in any direction be not less than 80% of the base shear calculated using the ELF method for regular structures nor 100% for irregular structures. This scaling requirement was introduced to ensure that assumptions used in building the analytical model does not result in excessively flexible representation of the structure and, consequently, an underestimate of the required strength.

3.5.7.4 Response History Analysis

Response history analysis is also permitted to be used for the design of any structure but, due to the added complexity, is seldom employed in practice except for special structures incorporating special base isolation or energy dissipation technologies. Either linear or nonlinear response history analysis is permitted to be used. When response history analysis is performed, input ground motion must consist of a suite of at least three pairs of orthogonal horizontal ground motion components, obtained from records of similar magnitude, source, distance, and site characteristics as the event controlling the hazard for the building's site. Each pair of orthogonal records must be scaled such that with a period range approximating the fundamental period of response of the structure, the square root of the sum of the squares of the orthogonal component ordinates envelopes 140% of the design response spectrum. Simple amplitude, rather than frequency domain scaling, is recommended. Actual records are preferred, though simulations may be used if a sufficient number of actual records representative of the design earthquake motion are not available. If a suite of less than seven records is used as input ground motion, the maximum of the response parameters (element forces and deformations) obtained from any of the records is used for design. If seven or more records are used, the mean values of the response parameters obtained from the suite of records may be used as design values. This requirement was introduced with the understanding that the individual characteristics of a ground motion record can produce significantly different results for some response quantities. It was hoped that this provision would encourage engineers to use larger suites of records and obtain an understanding of the variability associated with possible structural response.

When linear response history analyses are performed, the ground motion records, scaled as previously described, are further scaled by the quantity (I/R). The resulting member forces are combined with other loads, just as they would be if the ELF or response spectrum methods of analysis were performed.

When nonlinear response history analyses are performed, they must be used without further scaling. Rather than evaluating the strength of members using the standard load combinations considered with other analysis techniques, the engineer is required to demonstrate acceptable performance capability of the structure, given the predicted strength and deformation demands. The intention is that laboratory and other relevant data be used to demonstrate adequate behavior. This is a rudimentary introduction of performance-based design concepts, which will likely have significantly greater influence in future building codes.

3.5.8 Load Combinations and Strength Requirements

Structures must be proportioned with adequate strength to resist the forces predicted by the lateral seismic analysis together with forces produced by response to vertical components of ground shaking as well as dead and live loads. Unless nonlinear response history analysis is performed using ground motion records that include a vertical component of motion, the effects of vertical earthquake shaking are accounted for by the equation

$$E = Q_{\mathrm{E}} \pm 0.2 S_{\mathrm{DS}} D \tag{3.18}$$

where Q_{E} are the element forces predicted by the lateral seismic analysis, S_{DS} is the design spectral response acceleration at a 0.2 s response period, and D are the forces produced in the element by the structure's dead weight. The term $0.2S_{\mathrm{DS}}D$ represents the effect of vertical ground shaking response. For structures in zones of high seismicity, the term S_{DS} has a value approximating $1.0g$ and, therefore, the vertical earthquake effects are taken as approximately 20% increase or decrease in the dead load stress demands on each element. In fact, there are very few cases on record where structural collapse has been ascribed to the vertical response of a structure. This is probably because design criteria for vertical load resistance incorporate substantial factors of safety and also because most structures carry only a small fraction of their rated design live loads when they are subjected to earthquake effects. Therefore, most structures inherently have substantial reserve capacity to resist additional loading induced by vertical ground motion components. In recognition of this, most earlier codes neglected vertical earthquake effects. However, during the formulation of *ATC3.06*, it was felt to be important to acknowledge that ground shaking includes three orthogonal components. The resulting expression, which was somewhat arbitrary, ties vertical seismic forces to the short period design spectral response acceleration, as most structures are stiff vertically and have very short periods of structural response for vertical modes.

The earthquake forces on structural elements derived from Equation 3.18 are combined with dead and live loads in accordance with the standard strength level load combinations of *ASCE-7*. The pertinent load combinations are

$$Q = 1.4D \pm E \tag{3.19}$$

$$Q = 1.4D + 0.75(L + E) \tag{3.20}$$

where D, L, and E are the dead, live, and earthquake forces, respectively. Elements must then be designed to have adequate strength to resist these combined forces. The reduction factor of 0.75 on the combination of earthquake and live loads accounts for the low likelihood that a structure will be supporting full live load at the same time that it experiences full design earthquake shaking. An alternative set of load combinations is also available for use with design specifications that utilize allowable stress design formulations. These are essentially the same as Equations 3.19 and 3.20, except that the earthquake loads are further reduced by a factor of 1.4.

The *NEHRP Provisions* recognize that it is undesirable to allow some elements to experience inelastic behavior as they may be subject to brittle failure and in doing so compromise the ability of the structure to develop its intended inelastic response. The connections of braces to braced frames are an example of such elements. The *Provisions* also recognize that inelastic behavior in some elements, such as columns supporting discontinuous shear walls, could trigger progressive collapse of the structure. For these elements, the earthquake force E that must be used in the load combination (Equations 3.19 and 3.20) is given by the formula:

$$E = \Omega_0 Q_E \pm 0.2 S_{DS} D \tag{3.21}$$

where the term $0.2S_{DS}D$ continues to represent the effects of vertical ground shaking response and the term $\Omega_0 Q_E$ represents an estimate of the maximum force likely to be developed in the element as a result of lateral earthquake response, considering the inelastic response characteristics of the entire structural system. In Equation 3.20, the term $\Omega_0 Q_E$ need never be taken larger than the predicted force on the element derived from a nonlinear analysis or plastic mechanism analysis.

3.5.9 Drift Limitations

It is important to control lateral drift in structures because excessive drift can result in extensive damage to cladding and other nonstructural building components. In addition, excessive lateral drift can result in the development of P–Δ instability and collapse.

Lateral drift is evaluated on a story by story basis. Story drift, δ, is computed as the difference in lateral deflection at the top of a story and that at the bottom of the story, as predicted by the lateral analysis. If the lateral analysis was other than a nonlinear response history analysis, design story drift, Δ, is obtained from the computed story drift, δ, by the equation:

$$\Delta = C_d \delta \tag{3.22}$$

where C_d is the design coefficient previously discussed. The design interstory drift computed from Equation 3.22 must be less than a permissible amount, dependent on the SUG and structural system as given in Table 3.8.

The provisions require evaluation of potential P–Δ instability through consideration of the quantity θ given by the equation:

$$\theta = \frac{P_x \Delta}{V_x h_x C_d} \tag{3.23}$$

In this equation, P_x is the dead weight of the structure above story x, Δ is the design story drift, computed from Equation 3.22, V_x is the design story shear obtained from the lateral force analysis, h_x is the story height, and C_d is the coefficient previously discussed. If the quantity θ computed by this equation is found to be less than 0.1, P–Δ effects may be neglected. If the quantity θ is greater than 0.1, P–Δ effects must be directly considered in performing the LFA. If the quantity θ exceeds 0.3, the structure should be considered potentially unstable and must be redesigned.

This approach to P–Δ evaluation has remained essentially unchanged since its initial introduction in *ATC3.06*. It was introduced in that document as a placeholder, pending the development of a more

TABLE 3.8 Permissible Drift Limits[a]

	Seismic use group		
Structure	I	II	III
Structures other than masonry *shear wall* or masonry *wall*-frame *structures*, four stories or less in height with interior *walls*, *partitions*, ceilings, and exterior *wall* systems that have been designed to accommodate the *story* drifts	$0.025h_{sx}$[b]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever *shear wall structures*[c]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry *shear wall structures*	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
Masonry *wall*-frame *structures*	$0.013h_{sx}$	$0.013h_{sx}$	$0.010h_{sx}$
All other *structures*	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.

[b] h_{sx} is the story height below Level x.

[c] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

accurate method for evaluating drift-induced instability. Obvious deficiencies in this current approach include the fact that it evaluates drift effects at the somewhat artificial design-base shear levels. A more realistic evaluation would consider the actual expected lateral deformations of the structure as well as the yield level shear capacity of the structure at each story. As contained in the current provisions, evaluation of P–Δ effects seldom controls a structure's design.

3.5.10 Structural Detailing

Structural detailing is a critical feature of seismic-resistant design but is not generally specified by the *NEHRP Provisions*. Rather, the Provisions adopt detailing requirements contained in standard design specifications developed by the various materials industry associations including the American Institute of Steel Construction, the American Concrete Institute, the American Forest Products Association, and the Masonry Society. Other chapters in this handbook present the requirements of these various design standards.

Glossary

Attenuation — The rate at which earthquake ground motion decreases with distance.

Base shear — The total lateral force for which a structure is designed using equivalent lateral force techniques.

Characteristic earthquake — A relatively narrow range of magnitudes at or near the maximum that can be produced by the geometry, mechanical properties, and state of stress of a fault (Schwartz and Coppersmith 1987).

Completeness — Homogeneity of the seismicity record.

Cripple wall — A carpenter's term indicating a wood frame wall of less than full height T, usually built without bracing.

Critical damping — The value of damping such that free vibration of a structure will cease after one cycle (ccrit = 2 $m\omega$).

Damage — Permanent, cracking, yielding, or buckling of a structural element or structural assemblage.

Damping — Energy dissipation that occurs in a dynamically deforming structure, either as a result of frictional forces or structural yielding. Increased damping tends to reduce the amount that a structure responds to ground shaking.

Degradation — A behavioral mode in which structural stiffness or strength is reduced as a result of inelastic behavior.

Design (basis) earthquake — The earthquake (as defined by various parameters, such as PGA, response spectra, etc.) for which the structure will be, or was, designed.

Ductile detailing — Special requirements, such as for reinforced concrete and masonry, close spacing of lateral reinforcement to attain confinement of a concrete core, appropriate relative dimensioning of beams and columns, 135° hooks on lateral reinforcement, hooks on main beam reinforcement within the column, etc.

Ductile frames — Frames required to furnish satisfactory load-carrying performance under large deflections (i.e., ductility). In reinforced concrete and masonry this is achieved by ductile detailing.

Ductility factor — The ratio of the total displacement (elastic plus inelastic) to the elastic (i.e., yield) displacement.

Elastic — A mode of structural behavior in which a structure displaced by a force will return to its original state upon release of the force.

Fault — A zone of the earth's crust within which the two sides have moved — faults may be hundreds of miles long — from one to over one hundred miles deep, and not readily apparent on the ground surface.

Ground shaking — A random, rapid cyclic motion of the ground produced by an earthquake.
Hysteresis — A form of energy dissipation that is related to inelastic deformation of a structure.
Inelastic — A mode of structural behavior in which a structure, displaced by a force, exhibits permanent unrecoverable deformation.
Lateral force resisting system (LFRS) — A structural system for resisting horizontal forces due, for example, to earthquake or wind (as opposed to the vertical force resisting system, which provides support against gravity).
Liquefaction — A process resulting in a soil's loss of shear strength due to a transient excess of pore water pressure.
Magnitude — A unique measure of an individual earthquake's release of strain energy, measured on a variety of scales, of which the moment magnitude M_w (derived from seismic moment) is preferred.
Mass participation — That portion of total mass of a multidegree of freedom structure that is effective in a given mode of response.
MCE — Maximum considered earthquake — the earthquake intensity forming the basis for design in the *NEHRP Provisions*.
Mode shape — A deformed shape in which a structure can oscillate freely when displaced.
Natural mode — A characteristic dynamic property of a structure in which it will oscillate freely.
Nonductile frames — Frames lacking ducility or energy absorption capacity due to lack of ductile detailing — ultimate load is sustained over a smaller deflection (relative to ductile frames) and for fewer cycles.
Participation factor — A mathematical relationship between the maximum displacement of a multi-degree of freedom structure and a single degree of freedom structure.
Peak ground acceleration (PGA) — The maximum amplitude of recorded acceleration (also termed the ZPA, or zero period acceleration).
Period — The amount of time it takes a structure that has been displaced in a particular natural mode and then released to undergo one complete cycle of motion.
Pounding — The collision of adjacent buildings during an earthquake due to insufficient lateral clearance.
Response spectrum — A plot of maximum amplitudes (acceleration, velocity, or displacement) of a single degree of freedom oscillator (sdof), as the natural period of the sdof is varied across a spectrum of engineering interest (typically, for natural periods from 0.03 to 3 or more seconds, or frequencies of 0.3 to 30+ Hz).
Reverse fault — A fault that exhibits dip-slip motion, where the two sides are in compression and move away toward each other.
Seismic risk — The product of the hazard and the vulnerability (i.e., the expected damage or loss, or the full probability distribution).
Soft story — A story of a building signifiantly less stiff than adjacent stories (i.e., the lateral stiffness is 70% or less than that in the story above, or less than 80% of the average stiffness of the three stories above; BSSC 1994).
Spectral acceleration — The maximum response acceleration that a structure of given period will experience when subjected to a specific ground motion.
Spectral displacement — The maximum response displacement that a structure of given period will experience when subjected to a specific ground motion.
Spectral velocity — The maximum response velocity that a structure of given period will experience when subjected to a specific ground motion.
Spectrum amplification factor — The ratio of a response spectral parameter to the ground motion parameter (where parameter indicates acceleration, velocity, or displacement).
Viscous — A form of energy dissipation that is proportional to velocity.
Yielding — A behavioral mode in which a structural displacement increases under application of constant load.

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Further Reading

Chen and Scawthorn (2002) provide an extensive reference on earthquake engineering, while Naiem (2001) provides an excellent resource on seismic design. Both references have individual chapters on design of steel, wood, reinforced concrete, reinforced masonry, and precast structures, and also on nonstructural elements. SEAOC (1999), BSSC (1997), and SEAOC (1996) provide an excellent overview of the current state of seismic design requirements. U.S. Army (1992) and Dowrick (1987) are also useful, although a bit older. Some useful sources on seismic code provisions in countries other than the U.S. include *Earthquake Resistant Design Codes in Japan* (2000), Paz (1995), and IAEE (1996). The last is a comprehensive compendium of seismic regulations for over 40 countries, including *Eurocode 8* (the European Union's seismic provisions).