Understanding of the Theories and Practices of Earthquake-Resistant Design of Structures

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Introduction

In dealing with earthquakes, we must contend with appreciable probabilities that failure will occur soon. Otherwise, all the wealth of the world would prove insufficient to fill our needs: the most modest structures would be fortresses. We must also face uncertainty on a large scale, for it is our task to design engineering systems – about whose pertinent properties we know little – to resist future earthquakes and tidal waves – about whose characteristics we know even less. . . In a way, earthquake engineering is a cartoon.
Earthquake effects on structures systematically bring out the mistakes made in design and construction, even the minutest mistakes.

Several points are essential to an understanding of the theories and practices of earthquake-resistant design bear restating:

1. Ordinarily, a large earthquake produces the most severe loading that a building is expected to survive. The probability that *failure* will occur is very real and is greater than for other loading phenomena. Also, in the case of earthquakes, the definition of *failure* is altered to permit certain types of behavior and damage that are considered unacceptable in relation to the effects of other phenomena.

2. The levels of uncertainty are much greater than those encountered in the design of structures to resist other phenomena. This is in spite of the tremendous strides made since the Federal government began strongly supporting research in earthquake engineering and seismology following the 1964 Prince William Sound and 1971 San Fernando earthquakes. The high uncertainty applies both to knowledge of the loading function and to the resistance properties of the materials, members, and systems.

3. The details of construction are very important because flaws of no apparent consequence often will cause systematic and unacceptable damage simply because the earthquake loading is so severe, and an extended range of behavior is permitted.

The remainder of this course is devoted to a very abbreviated discussion of fundamentals that reflect the concepts on which earthquake-resistant design are based. When appropriate, important aspects of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* are mentioned and reference is made to particularly relevant portions of that document or the standards that are incorporated by reference. The *2020 Provisions* (FEMA, 2020a) are composed of three parts:

1) "Provisions", 2) "Commentary" and 3) "Resource Papers on Special Topics in Seismic Design." Part 1 states the intent and then cites ASCE/SEI 7-16 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2017) as the primary reference. The remainder of Part 1 contains recommended changes to update ASCE/SEI 7-16; the recommended changes include commentary on each specific recommendation. All three parts are referred to herein as the *Provisions,* but where pertinent the specific part is referenced, and ASCE/SEI 7-16 is referred to as the *Standard*. ASCE/SEI 7-16 itself refers to several other standards for the seismic design of structures composed of specific materials and those standards are essential elements to achieve the intent of the *Provisions*.

1. Earthquake Phenomena

According to the most widely held scientific belief, most earthquakes occur when two segments of the earth's crust suddenly move in relation to one another. The surface along which movement occurs is known as a fault. The sudden movement releases strain energy and causes seismic waves to propagate through the crust surrounding the fault. These waves cause the surface of the ground to shake violently, and it is this ground shaking that is the principal concern of structural engineering to resist earthquakes.

Earthquakes have many effects in addition to ground shaking. For various reasons, many of the other effects generally are not major considerations in the design of buildings and similar structures.
For example, seismic sea waves or tsunamis can cause very forceful flood waves in coastal regions, and seiches (long-period sloshing) in lakes and inland seas can have similar effects along shorelines.
These are outside the scope of the *Provisions*. The devasting tsunamis accompanying the 2004 Sumatra-Andaman and the 2010 Tohoku Earthquakes stimulated the development of methods to design structures to resist such hydrodynamic forces, and ASCE/SEI 7-16 includes a chapter devoted to that effect. Long-period sloshing of the liquid contents of tanks is addressed by the *Provisions*.

Abrupt ground displacements occur where a fault intersects the ground surface. (This commonly occurs in California earthquakes but did not occur in the historic Charleston, South Carolina earthquake or the very large New Madrid, Missouri earthquakes of the nineteenth century.) Mass soil failures such as landslides, liquefaction, and gross settlement result from ground shaking on susceptible soil formations. Once again, design for such events is specialized, and it is common to locate structures so that mass soil failures and fault rupture are of no major consequence to their performance. Modifying soil properties to protect against liquefaction is one important exception; large portions of a few metropolitan areas with the potential for significant ground shaking are susceptible to liquefaction. Lifelines that cross faults require special design beyond the scope of the *Provisions*. The structural loads specified in the *Provisions* are based solely on ground shaking; they do not provide for ground failure. Resource Paper 12 ("Evaluation of Geologic Hazards and Determination of Seismic Lateral Earth Pressures") in Part 3 of the 2009 *Provisions* (FEMA, 2009) includes a description of current procedures for predicting seismic-induced slope instability, liquefaction and surface fault rupture. Selected portions of that work are now included in the *Provisions*.

Nearly all large earthquakes are *tectonic* in origin. They are associated with movements of and strains in large segments of the earth's crust, called *plates*, and virtually all such earthquakes occur at or near the boundaries of these plates. This is the case with earthquakes in the far western portion of the United States, where two very large plates, the North American continent and the Pacific basin, come together. In the central and eastern United States, however, earthquakes are not associated with such a plate boundary, and their causes are not as completely understood. This factor, combined with the smaller amount of data about central and eastern earthquakes (because of their infrequency), means that the uncertainty associated with earthquake loadings is higher in the central and eastern portions of the nation than in the West. Even in the west, the uncertainty (when considered as a fraction of the predicted level) about the hazard level is probably greater in areas where the mapped hazard is low than in areas where the mapped hazard is high.

Two basic data sources are used in establishing the likelihood of earthquake ground shaking, or seismicity, at a given location. The first is the historical record of earthquake effects and the second is the geological record of earthquake effects. Given the infrequency of major earthquakes, there is no place in the United States where the historical record is long enough to be used as a reliable basis for earthquake prediction – certainly not as reliable as with other phenomena such as wind and snow. Even on the eastern seaboard, the historical record is too short to justify sole reliance on the historical record. Thus, the geological record is essential. Such data requires very careful interpretation, but they are used widely to improve knowledge of seismicity. Geological data have been developed for many locations as part of the nuclear power plant design process. Overall, there is more geological data available for the far western United States than for other regions of the country. Both sets of data have been considered in the *Provisions* seismic ground shaking maps. In recent years, data from earthquakes associated with pumping fluid into deep wells have also been considered in understanding the geologic procedures.

The amplitude of earthquake ground shaking diminishes with distance from the source, and the rate of attenuation is less for lower frequencies of motion than for higher frequencies. This effect is captured by the fact that the *Provisions* specify response acceleration parameters at 22 frequencies of vibration to define the hazard of seismic ground shaking for structures. They are based on a statistical analysis of the database of seismological information. The *Provisions* provide one additional parameter for the definition of response to ground shaking, *TL*. It defines an important transition point for long period (low frequency) behavior; it is not based upon as robust of an analysis as the other parameters.

The *Commentary* provides a more thorough discussion of the development of maps, their probabilistic basis, the necessarily crude lumping of parameters and other related issues. Prior to its 1997 edition, the basis of the *Provisions* was to "minimize the hazard to life…" at the design earthquake motion, which was defined as having a 10 percent probability of being exceeded in a 50- year reference period (FEMA, 1995). As of the 1997 edition (FEMA, 1997), the basis became to avoid *structural collapse* at the maximum considered earthquake (MCE) ground motion, which is defined as having a 2 percent probability of being exceeded in a 50-year reference period. In the 2009 edition of the *Provisions* the design basis was refined to target a 1% probability of structural collapse for ordinary buildings in a 50-year period. The MCE ground motion has been adjusted to deliver this level of risk combined with a 10% probability of collapse should the MCE ground motion occur. This new approach incorporates a fuller consideration of the nature of the seismic hazard at a location than was possible with the earlier definitions of ground shaking hazard, which were tied to a single level of probability of ground shaking occurrence.

The nature of the uncertainty in earthquake occurrence and in ground shaking amplitude combine to predict very high ground motions near faults that produce large earthquakes relatively frequently. Empirical evidence of building performance in past earthquakes indicates that design for such extreme motions is not necessary. Consequently, when the MCE concept was introduced, the *Provisions* included a semi-deterministic upper bound on the accelerations produced by the purely probabilistic method. The concept used was to combine the occurrence of a reasonable upper bound earthquake at the known fault location with a somewhat conservative estimate (mean plus one standard deviation) of the ground shaking at a site. The details of this method have evolved in subsequent editions of the *Provisions*, but the philosophical basis remains the same.

2. Structural Response to Ground Shaking

The first important difference between structural response to an earthquake and response to most other loadings is that the earthquake response is *dynamic*, not *static*. For most structures, even the response to wind is essentially static. Forces within the structure are due almost entirely to pressure loading rather than the acceleration of the mass of the structure. But with earthquake ground shaking, the above ground portion of a structure is not subjected to any applied force. The stresses and strains within the superstructure are created entirely by its dynamic response to the movement of its base, the ground. Even though the most used design procedure resorts to the use of a concept called the equivalent static force for actual calculations, some knowledge of the theory of vibrations of structures is essential.

Response Spectra

Figure 1 shows accelerograms, records of the acceleration at one point along one axis, for several representative earthquakes. Note the erratic nature of the ground shaking and the different characteristics of the different accelerograms. Precise analysis of the elastic response of an ideal structure to such a pattern of ground motion is possible; however, it is not commonly done for ordinary structures. The increasing power and declining cost of computational aids are making such analyses more common, but, at this time, only a small minority of structures designed across the country are analyzed for specific response to a specific ground motion.

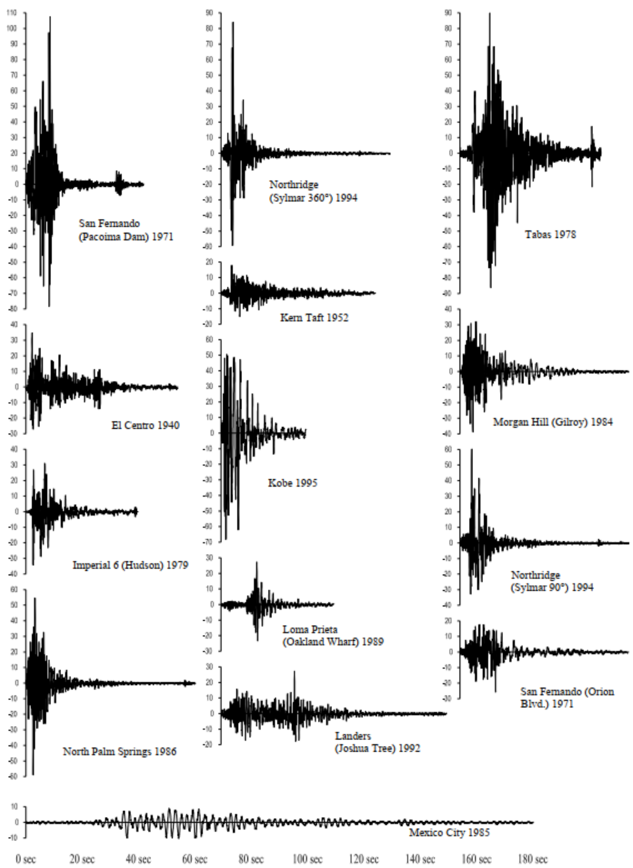

Figure 1. Earthquake Ground Acceleration in Epicentral Regions. Note: All accelerograms are plotted to the same scale for time and acceleration – the vertical axis is % gravity). Great earthquakes extend for much longer periods of time.)

Figure 2 shows further detail developed from an accelerogram. Part (a) shows the ground acceleration along with the ground velocity and ground displacement derived from it. Part (b) shows the acceleration, velocity, and displacement for the same event at the roof of the building located where the ground motion was recorded. Note that the peak values are larger in the diagrams of Figure 2(b) (the vertical scales are essentially the same). This increase in response of the structure at the roof level over the motion of the ground itself is known as dynamic amplification. It depends very much on the vibrational characteristics of the structure and the characteristic frequencies of the ground shaking at the site.

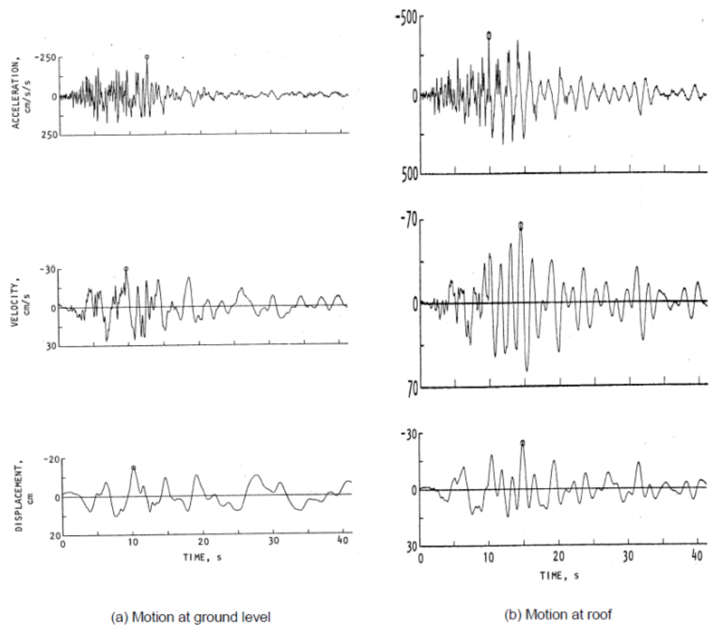

Figure 2. Holiday Inn Ground and Building Roof Motion During the M6.4 1971 San Fernando Earthquake: (a) North-South Ground Acceleration, Velocity and Displacement and (b) North-South Roof Acceleration, Velocity and Displacement (Housner and Jennings, 1982).

The building was a seven-story, reinforced concrete frame, approximately five miles from the closest portion of the causative fault.

In design, the response of a specific structure to an earthquake is ordinarily estimated from a design response spectrum such as what is specified in the *Provisions*. The first step in creating a design response spectrum is to determine the maximum response of a given structure to a specific ground motion (see the maximum response points denoted by the circles in Figure 2b). The underlying theory is based entirely on the response of a single-degree-of-freedom oscillator, such as a simple one-story frame with the mass concentrated at the roof. The vibrational characteristics of such a simple oscillator may be reduced to two: the natural period1 and the amount of damping. By recalculating the record of response versus time to a specific ground motion for a wide range of natural periods and for each of a set of common amounts of damping, the family of response spectra for one ground motion may be determined. It is simply the plot of the maximum value of response for each combination of period and damping.

Figure 3 shows such a result for the ground motion of Figure 2(a) and illustrates that the erratic nature of ground shaking leads to a response that is very erratic in that a slight change in the natural period of vibration brings about a very large change in response. The figure also illustrates the significance of damping. Different earthquake ground motions lead to response spectra with peaks and valleys at different points with respect to the natural period. Thus, computing response spectra for several different ground motions and then averaging them, based on some normalization for different amplitudes of shaking, will lead to a smoother set of spectra. Such smoothed spectra are an important step in developing a design spectrum.

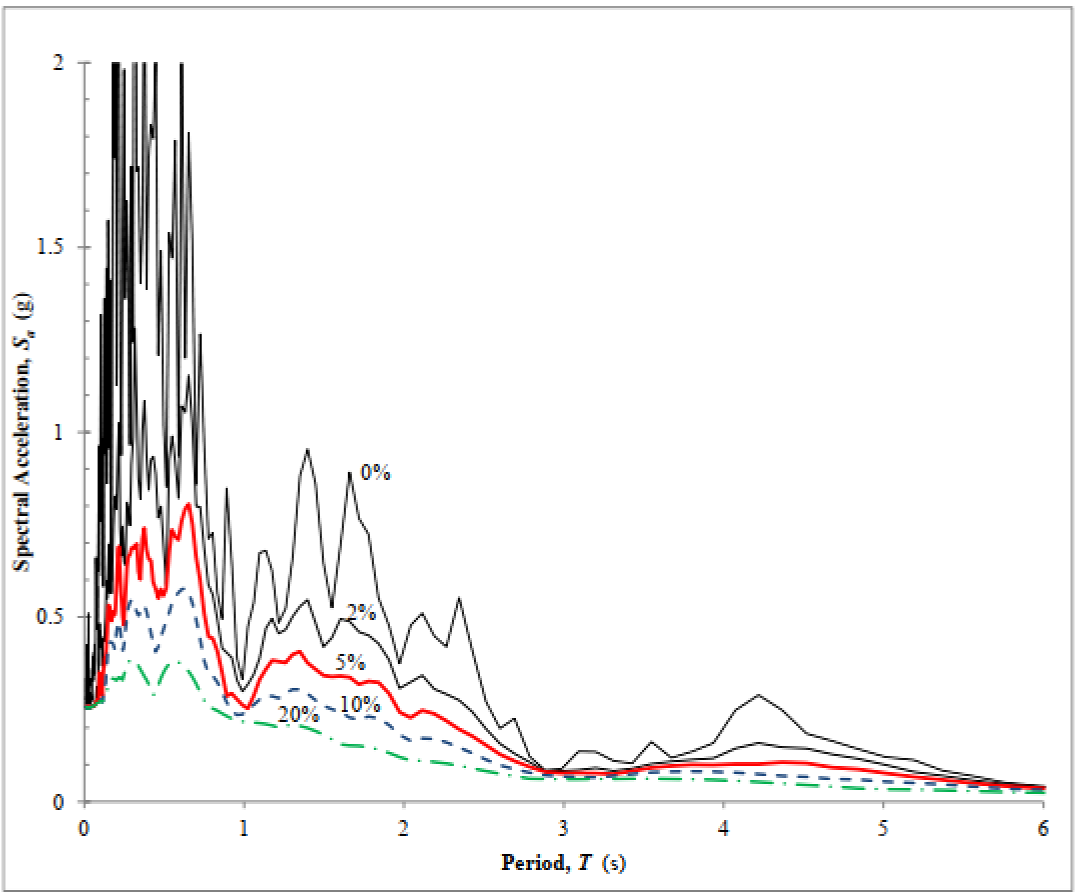

Figure 3. Response Spectrum of North-South Ground Acceleration (0%, 2%, 5%, 10%, 20% of Critical Damping) Recorded at the Holiday Inn, Approximately Five miles from the Causative Fault in the 1971 San Fernando Earthquake

Much of the literature on dynamic response is written in terms of frequency rather than period. The cyclic frequency (cycles per second, or Hz) is the inverse of period. Mathematically it is often convenient to use the angular frequency expressed as radians per second rather than Hz. The conventional symbols used in earthquake engineering for these quantities are ***T*** for period (seconds per cycle), ***f*** for cyclic frequency (Hz) and $\boldsymbol{\omega}$ for angular frequency (radians per second). The word frequency is often used with no modifier; be careful with the units.

Figure 4 is an example of an averaged spectrum. Note that acceleration, velocity, or displacement may be obtained from Figure 3 or 4 for a structure with a known period and damping.

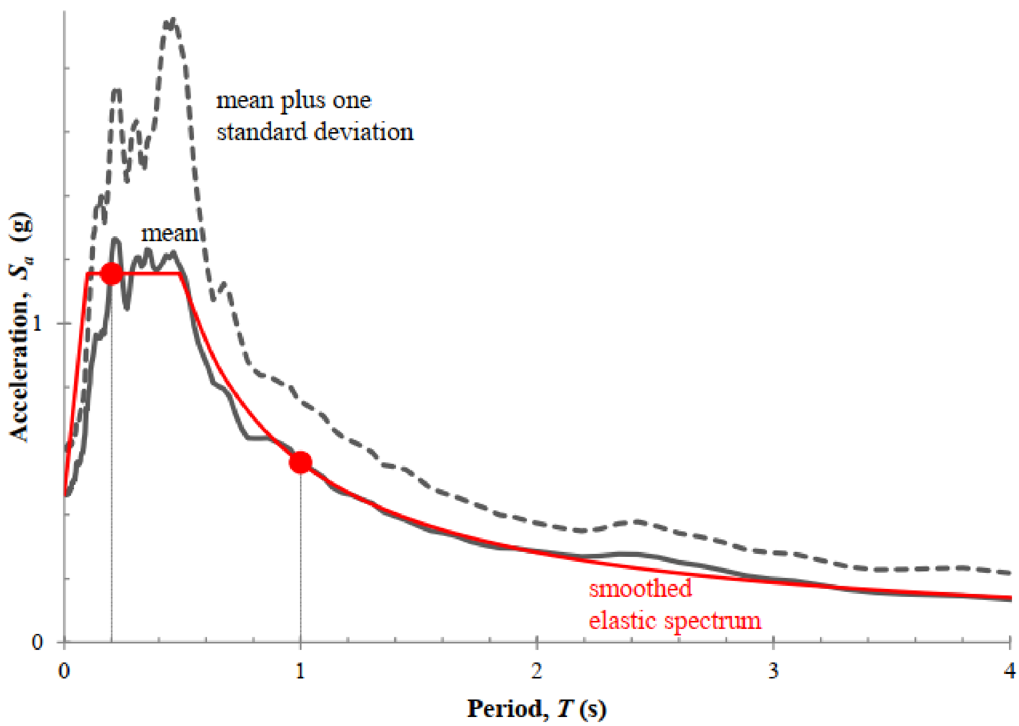

Figure 4. Averaged Spectrum. Note: In this case, the statistics are for seven ground motions representative of the de-aggregated hazard at a particular site.

Prior to the 1997 edition of the *Provisions*, the maps that characterized the ground shaking hazard were plotted in terms of peak ground acceleration (at period $T = 0$), and design response spectra were created using expressions that amplified (or de-amplified) the ground acceleration as a function of period and damping. With the introduction of the MCE maps in the 1997 edition, this procedure changed. Those maps presented spectral response accelerations at two periods of vibration, 0.2 and 1.0 second, and the design response spectrum was computed more directly, as implied by the smooth line in Figure 4. This has removed a portion of the uncertainty in predicting response accelerations.

The ground motions in the 2020 *Provisions* are given as spectral response accelerations at 22 periods from zero to 10 seconds. The shape of the spectrum varies from one location to another, but the two spectral ordinates for construction of the familiar spectral shape are also given for conventional analysis. Figure 5 shows the two spectra for a location in Southern California.

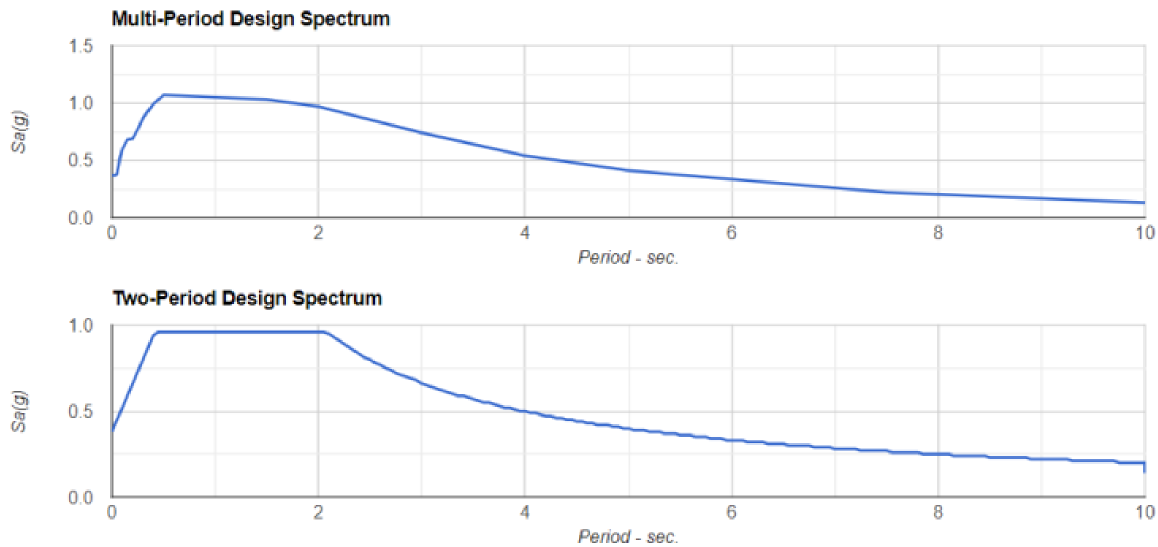

Figure 5. Comparison of the Multi-period Design Spectrum with the Two-period Spectrum from the 2020 Provisions for a Site in Southern California

Few structures are simple enough to vibrate as a single-degree-of-freedom system. The principles of dynamic modal analysis, however, allow a reasonable approximation of the maximum response of a multi-degree-of-freedom oscillator, such as a multistory building, if many specific conditions are met. The procedure involves dividing the total response into several natural modes, modeling each mode as an equivalent single-degree-of-freedom oscillator, determining the maximum response for each mode from a single-degree-of-freedom response spectrum and then estimating the maximum total response by statistically summing the responses of the individual modes. The *Provisions* does not require consideration of all possible modes of vibration for most buildings because the contribution of the higher modes (lower periods) to the total response is relatively minor.

The soil at a site has a significant effect on the characteristics of ground motion and, therefore, on the structure's response. Especially at low amplitudes of motion and at longer periods of vibration, soft soils amplify the motion at the surface with respect to bedrock motions. This amplification is diminished somewhat, especially at shorter periods as the amplitude of basic ground motion increases due to yielding in the soil. The *Provisions* accounts for this effect by providing amplifiers that are to be applied to the spectral accelerations for various classes of soils. The site classes are based upon the velocity of a shear wave passing through the soil, averaged over the top 100 feet (30 meters). The amount of amplification depends on both that average velocity and the amplitude of the motion in rock. Thus, very different design response spectra are specified depending on the type of soil(s) beneath the structure. The *Commentary* (Part 2) contains a thorough explanation of this feature.

Inelastic Response

The preceding discussion assumes elastic behavior of the structure. The principal extension beyond ordinary behavior referenced at the beginning of this chapter is that structures are permitted to strain beyond the elastic limit in responding to earthquake ground shaking. This is dramatically different from the case of design for other types of loads in which stresses, and therefore strains, are not permitted to approach the elastic limit. The reason is economic. Figure 3 shows a peak acceleration response of about 1.0 g (the acceleration due to gravity) for a structure with moderately low damping – for only a moderately large earthquake! Even structures that resist lateral forces will have a static lateral strength of only 20 to 40 percent of gravity.

The dynamic nature of earthquake ground shaking means that a large portion of the shaking energy can be dissipated by inelastic deformations if the structure is ductile and some damage to the structure is accepted. Figure 6 will be used to illustrate the significant difference between wind and seismic effects. Figure 6 (a) would represent a cantilever beam if the load *W* were small and a column if *We* were large. Wind pressures create a force on the structure, which in turn produces a displacement. The force is the independent variable, and the displacement is the dependent result. Earthquake ground motion creates displacement between the base and the mass, which in turn produces an internal force. The displacement is the independent variable, and the force is the dependent result. Two graphs are plotted with the independent variables on the horizontal axis and the dependent response on the vertical axis. Thus, Part (b) of Figure 6 is characteristic of the response to forces such as wind pressure (or gravity weight), while Part (c) is characteristic of induced displacements such as earthquake ground shaking (or foundation settlement).

Note that the ultimate resistance (*Hu*) in a force-controlled system is marginally larger than the yield resistance (*Hy*), while the ultimate displacement (Δu) in a displacement-controlled system is much larger than the yield displacement (Δy). The point being made with the figures is that ductile structures could resist displacements much larger than those that first cause yield. Thus, ductility is a much more important property when the demand is displacement than when the demand is forced.

The degree to which a member or structure may deform beyond the elastic limit is usually referred to as ductility. Different materials and different arrangements of structural members lead to different ductility. Response spectra may be calculated for oscillators with different levels of ductility. At the risk of oversimplification, the following conclusions may be drawn:

1. For structures with very long natural periods, the acceleration response is reduced by a factor equivalent to the ductility ratio (the ratio of maximum usable displacement to effective yield displacement – note that this is displacement and not strain).

2. For structures with very short natural periods, the acceleration response of the ductile structure is essentially the same as that of the elastic structure, but the displacement is increased.

3. For intermediate periods (which applies to nearly all buildings), the acceleration response is reduced, but the displacement response is generally about the same for the ductile structure as for the elastic structure strong enough to respond without yielding

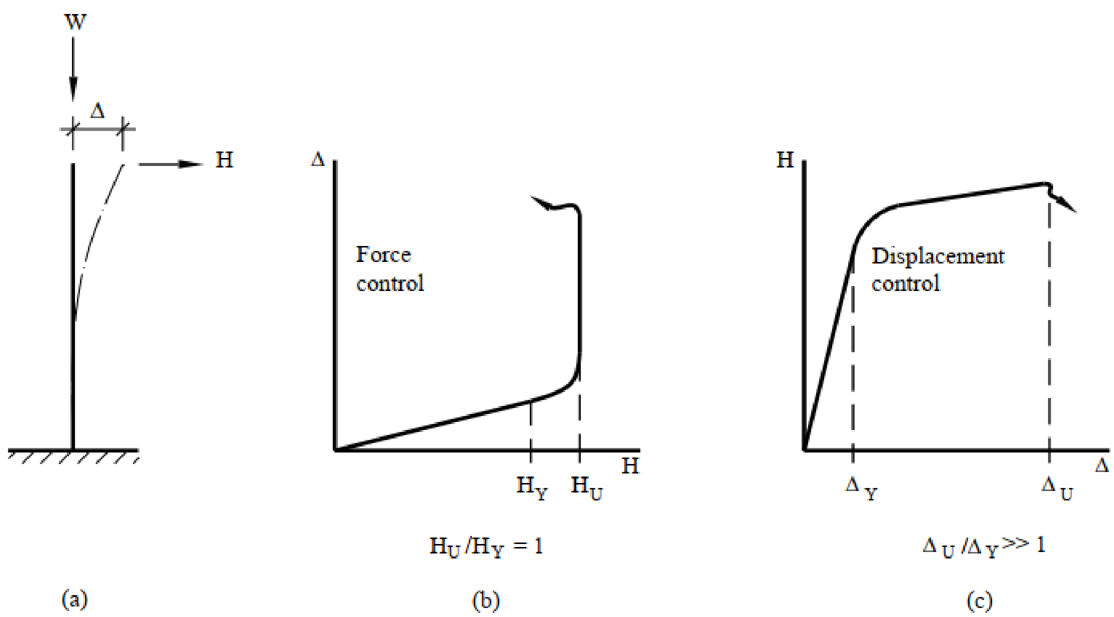

Figure 6. Force Controlled Resistance Versus Displacement Controlled Resistance (after Housner and Jennings 1982)

Note: In Part (b), force H is the independent variable. As H increases, the displacement increases until the yield point stress is reached. If H is given an additional increment (about 15 percent) a plastic hinge forms, giving large displacements. For this kind of system, the force producing yield point stress is close to the force producing collapse. The ductility does not produce a large increase in load capacity, although in highly redundant structures the increase is more than illustrated for this very simple structure. In Part (c) the displacement is the independent variable. As the displacement is increased, the base moment increases until the yield point is reached. As the displacement increases still more, the resistance (H) increases only a small amount. For a highly ductile element, the displacement can be increased 10 to 20 times the yield point displacement before the system collapses under the weight W. (As W increases, this ductility decreases dramatically.) During an earthquake, the oscillator is excited into vibrations by ground motion, and it behaves essentially as a displacement-controlled system and can survive displacements much beyond the yield point. This explains why ductile structures can survive ground shaking that produces displacements much greater than yield point displacement.

Inelastic response is quite complex. Earthquake ground motions involve a significant number of reversals and repetitions of the strains. Therefore, observation of the inelastic properties of a material, member, or system under a monotonically increasing load until failure can be very misleading. Cycling deformation can cause degradation of strength, stiffness, or both. Systems that have a proven capacity to maintain a stable resistance to many cycles of inelastic deformation are allowed to exercise a greater portion of their ultimate ductility in designing for earthquake resistance. This property is often referred to as toughness, but this is not the same as the classic definition used in mechanics of materials, which is the strain energy to failure under monotonic loading.

Most structures are designed for seismic response using a linear elastic analysis with the strength of the structure limited by the strength at its critical location. Most structures possess enough complexity so that the peak strength of a ductile structure is not accurately captured by such an analysis. Figure 7 shows the load versus displacement relation for a simple frame. Yield must develop at four locations before the peak resistance is achieved. The margin from the first yield to the peak strength is referred to as overstrength, and it plays a significant role in resisting strong ground motion. Note that a few key design standards (for example, American Concrete Institute (ACI) 318 for the design of concrete structures) do allow for some redistribution of internal forces from the critical locations based upon ductility; however, the redistributions allowed therein are minor compared to what occurs in response to strong ground motion. Many types of structures, particularly buildings also possess additional overstrength from the resistance to lateral displacement provided by structural elements not deemed to be a part of the seismic-resisting system and by nonstructural elements, such as cladding.

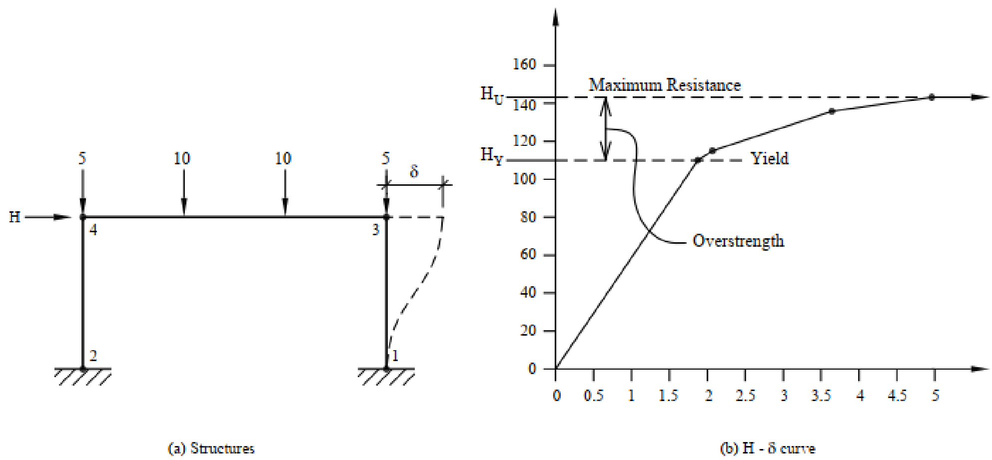

Figure 7. Initial Yield Load and Failure for a Ductile Portal Frame

Note: The margin from initial yield to failure (mechanism in this case) is known as overstrength.

To summarize, the characteristics important in determining a building's seismic response are natural period, damping, ductility, stability of resistance under repeated reversals of inelastic deformation and overstrength. The natural frequency is dependent on the mass and stiffness of the building. Using the *Provisions*, the designer calculates, or at least approximately, the natural period of vibration (the inverse of natural frequency). Damping, ductility, toughness and overstrength depend primarily on the type of building system but not the building's size or shape. Recent studies have shown that the total deformation capacity of the structure may be a more useful parameter than a ductility ratio to characterize a structure's resistance to collapse, but quantification of performance based on that parameter is still a research topic. Three coefficients – *R*, *Cd*, and *Ω0* – are provided to encompass damping, ductility, stability of resistance and overstrength. *R* is intended to be a conservatively low estimate of the reduction of acceleration response in a ductile system from that for an elastic oscillator with a certain level of damping. It is used to compute the required strength. Computations of displacement based upon ground motion reduced by the factor *R* will underestimate the actual displacements. *Cd* is intended to be a reasonable mean for the amplification necessary to convert the elastic displacement response computed for the reduced ground motion to actual displacements. *Ω0* is intended to deliver a reasonably high estimate of the peak force that would develop in the structure. Sets of *R*, *Cd*, and *Ω0* are specified in the *Provisions* for the most common structural materials and systems.

Building Materials

The following brief comments about building materials and systems are included as general guidelines only, not for specific application.

WOOD

Timber structures nearly always resist earthquakes very well, even though wood is a brittle material as far as tension and flexure are concerned. It has some ductility in compression (generally monotonic), and its strength is significantly higher for brief loadings, such as in an earthquake, than for long term loads. Conventional timber structures (plywood, oriented strand board, or board sheathing on wood framing) possess much more ductility than the basic material primarily because the nails and other steel connection devices yield, and the wood compresses against the connector.

These structures also possess a much higher degree of damping than the damping that is assumed in developing the basic design spectrum. Much of this damping is caused by slips at the connections. Light-framed wood construction also usually has significant overstrength from nonstructural sheathing material on walls and partitions. The increased strength, connection ductility, and high damping combine to give timber structures a large reduction from elastic response to design level.

This large reduction should not be used if the strength of the structure is controlled by bending or tension of the gross timber cross sections. The large reduction in acceleration combined with the lightweight timber structures make them very efficient regarding earthquake ground shaking when they are properly connected. This is confirmed by their generally good performance in earthquakes.

Capacities and design and detailing rules for wood elements of seismic force-resisting systems are now found in the *Special Design Provisions for Wind and Seismic* (AWC, 2020) supplement to the *National Design Specification for Wood Construction* (AWC, 2017).

STEEL

Steel is the most ductile of the common building materials. The moderate-to-large reduction from elastic response to design response allowed for steel structures is primarily a reflection of this ductility and the stability of the resistance of steel. Members subject to buckling (such as bracing) and connections subject to brittle fracture (such as partial penetration welds under tension) are much less ductile and are addressed in the *Provisions* in various ways. Defects, such as stress concentrations and flaws in welds, also affect earthquake resistance, as demonstrated in the Northridge earthquake. The basic and applied research program that grew out of that experience has greatly increased knowledge of how to avoid low ductility details in steel construction. Capacities and design and detailing rules for seismic design of hot-rolled structural steel are found in the *Seismic Provisions for Structural Steel Building*s (AISC, 2016) and similar provisions for cold-formed steel are found in the *North American Standard for Seismic Design* (AISI, 2021).

REINFORCED CONCRETE

Reinforced concrete achieves ductility through careful limits on steel in tension and concrete in compression. Reinforced concrete beams with common proportions can possess ductility under monotonic loading even greater than common steel beams, in which local buckling is usually a limiting factor. Providing stability of the resistance to reversed inelastic strains, however, requires special detailing. Thus, there is a wide range of reduction factors from elastic response to design response depending on the detailing for stable and assured resistance. The 2020 *NEHRP Provisions Commentary* and the commentary with the ACI 318 standard *Building Code Requirements for Structural Concrete* (ACI, 2019) explain how to design to control premature shear failures in members and joints, buckling of compression bars, concrete compression failures (through confinement with transverse reinforcement), the sequence of plastification and other factors, which can lead to large reductions from the elastic response.

MASONRY

Masonry is a more complex material than those mentioned above and less is known about its inelastic response characteristics. For certain types of members (such as pure cantilever shear walls), reinforced masonry behaves in a fashion like reinforced concrete. The nature of masonry construction, however, makes it difficult, if not impossible, to take some of the steps (e.g., confinement of compression members) used with reinforced concrete to increase ductility, and stability. Further, the discrete differences between mortar, grout and the masonry unit create additional failure phenomena. Thus, the response reduction factors for the design of reinforced masonry are not quite as large as those for reinforced concrete. Unreinforced masonry possesses little ductility or stability, except for rocking of masonry piers on a firm base and very little reduction from the elastic response is permitted. Capacities and design and detailing rules for seismic design of masonry elements are contained within The Masonry Society (TMS) 402 standard *Building Code Requirements for Masonry Structures*.

PRECAST CONCRETE

Precast concrete can behave quite similarly to reinforced concrete, but it also can behave quite differently. The connections between pieces of precast concrete commonly are not as strong as the members being connected. Clever arrangements of connections can create systems in which yielding under earthquake motions occurs away from the connections, in which case the similarity to reinforced concrete is very real. Some carefully detailed connections also can mimic the behavior of reinforced concrete. Many common connection schemes, however, will not do so. Successful performance of such systems requires that the connections perform in a ductile manner. This requires some extra effort in design, but it can deliver successful performance. As a point of reference, the most common wood seismic-resisting systems perform well yet have connections (nails) that are significantly weaker than the connected elements (structural wood panels). Prior editions of the *Provisions* introduced advances in seismic design of precast system through important Part 3 papers. The advances have found their way into ASCE/SEI 7 and ACI 318. There are also supplemental ACI standards for specialized seismic force-resisting systems of precast concrete.

COMPOSITE STEEL AND CONCRETE

Reinforced concrete is a composite material. In the context of the *Provisions*, *composite* is a term reserved for structures with elements consisting of structural steel and reinforced concrete acting in a composite manner. These structures generally are an attempt to combine the most beneficial aspects of each material. Capacities and design and detailing rules are found in the *Seismic Provisions for Structural Steel Buildings* (AISC Standard 341).

Building Systems

Three basic lateral-load-resisting elements – walls, braced frames, and unbraced frames (moment resisting frames) – are used to build a classification of structural types in the *Provisions*. Unbraced frames generally are allowed greater reductions from elastic response than walls and braced frames. In part, this is because frames are more redundant, having several different locations with approximately the same stress levels and common beam-column joints frequently exhibit an ability to maintain a stable response through many cycles of reversed inelastic deformations. Systems using connection details that have not exhibited good ductility and toughness, such as unconfined concrete and the welded steel joint used before the Northridge earthquake, are penalized: the *R* factors permit less reduction from elastic response.

Connection details often make the development of ductility difficult in braced frames, and buckling of compression members also limits their inelastic response. The actual failure of steel bracing often occurs because local buckling associated with overall member buckling frequently leads to locally high strains that then lead to brittle fracture when the member subsequently approaches yield in tension. Eccentrically braced steel frames and new proportioning and detailing rules for concentrically braced frames have been developed to overcome these shortcomings. But the newer and more popular bracing system is the buckling-restrained braced frame. This new system has the advantages of a special steel concentrically braced frame, but with performance that is superior as brace buckling is controlled to preserve ductility. Design provisions appear in the *Seismic Provisions for Structural Steel Buildings* (AISC Standard 341).

Shear walls that do not bear gravity load are allowed a greater reduction than walls that are load bearing. Redundancy is one reason; another is that axial compression generally reduces the flexural ductility of concrete and masonry elements (although small amounts of axial compression usually improve the performance of materials weak in tension, such as masonry and concrete). The 2010 earthquake in Chile has led to improvements in understanding and design of reinforced concrete shear wall systems, because of the large number of significant concrete shear wall buildings subjected to strong shaking in that earthquake. Systems that combine different types of elements generally allowed greater reductions from elastic response because of redundancy.

Redundancy is frequently cited as a desirable attribute for seismic resistance. A quantitative measure of redundancy is included in the *Provisions* to prevent the use of large reductions from elastic response in structures that possess very little redundancy. Only two values of the redundancy factor, ρ, are defined: 1.0 and 1.3. The penalty factor of 1.3 is placed upon systems that do not possess some elementary measures of redundancy based on explicit consideration of the consequence of failure of a single element of the seismic force-resisting system. A simple, deemed-to-comply exception is provided for certain structures.

Supplementary Elements Added to Improve Structural Performance

The *Standard* includes provisions for the design of two systems to significantly alter the response of the structure to ground shaking. Both have specialized rules for response analysis and design detailing.

Seismic isolation involves the placement of specialized bearings with low lateral stiffness and large lateral displacement capacity between the foundation and the superstructure. It is used to substantially increase the natural period of vibration and thereby decrease the acceleration response of the structures. (Recall the shape of the response spectrum in Figure 4; the acceleration response beyond a threshold period is roughly proportional to the inverse of the period). Seismic isolation is becoming increasingly common for structures in which superior performance is necessary, such as major hospitals and emergency response centers. Such structures are frequently designed with a stiff superstructure to control story drift, and isolation makes it feasible to design such structures for lower total lateral force. The design of such systems requires a conservative estimate of the likely deformation of the isolator. The early provisions for that factor were a precursor of the changes in ground motion mapping implemented in the 1997 *Provisions*.

Added damping involves the placement of specialized energy dissipation devices within stories of the structure. The devices can be like a large shock absorber, but other technologies are also available. Added damping is used to reduce the structural response and the effectiveness of increased damping can be seen in Figure 3. It is possible to reach effective damping levels of 20 to 30 percent of critical damping, which can reduce response by factors of 2 or 3. The damping does not have to be added in all stories; in fact, it is common to add damping at the isolator level of seismically isolated buildings.

Isolation and damping elements require extra procedures for analysis of seismic response. Both also require considerations beyond common building construction to assure quality and durability.

3. Engineering Philosophy

The *Commentary*, under "Intent," states:
"The primary intent of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* is to prevent, for ordinary buildings and structures, serious injury and life loss caused by damage from earthquake ground shaking and ground failure. Most earthquake injuries and deaths are caused by structural collapse; therefore, the major thrust of the *Provisions* is to prevent collapse for very rare, intense ground motion, termed the risk targeted maximum earthquake (MCER) motion. Additional objectives to preserve means of egress, maintain functionality of critical or essential facilities following major earthquakes, and to reduce damage costs, where practicable, are addressed as corollaries to the primary intent."

The *Provisions* states:
"The degree to which these objectives can be achieved depends on several factors including structural framing type, building configuration, structural and nonstructural materials and details, and overall quality of design and construction. In addition, large uncertainties as to the intensity and duration of shaking and the possibility of unfavorable response of a small subset of buildings or other structures may prevent full realization of the intent."

At this point, it is worth recalling the criteria mentioned earlier in describing the risk-targeted ground motions used for design. The probability of structural collapse due to ground shaking is not zero. One percent in 50 years is a higher failure rate than is currently considered acceptable for buildings subject to other natural loads, such as wind and snow. The reason is as stated in the quote at the beginning of this chapter "…all the wealth of the world would prove insufficient…" Damage is to be expected when an earthquake equivalent to the design earthquake occurs. (The "design earthquake" is currently taken as two-thirds of the MCE ground motion). Some collapse is to be expected when and where ground motion equivalent to the MCE ground motion occurs.

The basic structural criteria are strength, stability, and distortion. The yield-level strength provided must be at least that required by the design spectrum (which is reduced from the elastic spectrum as described previously). Structural elements that cannot be expected to perform in a ductile manner are to have greater strength, which is achieved by applying the $\Omega 0$ amplifier to the design spectral response. The stability criterion is imposed by amplifying the effects of lateral forces for the destabilizing effect of lateral translation of the gravity weight (the P-Delta effect). The distortion criterion is a limit on story drift and is calculated by amplifying the linear response to the (reduced) design spectrum by the factor Cd to account for inelastic behavior.

Yield-level strengths for steel and concrete structures are easily obtained from common design standards. The most common design standards for timber and masonry are based on allowable stress concepts that are not consistent with the basis of the reduced design spectrum. Although strength-based standards for both materials have been introduced in recent years, the engineering profession has not yet embraced these new methods. In the past, the *Provisions* stipulated adjustments to common reference standards for timber and masonry to arrive at a strength level equivalent to yield and compatible with the basis of the design spectrum. Most of these adjustments were simple factors to be applied to conventional allowable stresses. With the deletion of these methods from the *Provisions*, other methods have been introduced into model building codes and the ASCE standard, *Minimum Design Loads for Buildings and Other Structures* to factor downward the seismic load effects based on the *Provisions* for use with allowable stress design methods.

The *Provisions* recognizes that the risk presented by a particular building is a combination of the seismic hazard at the site and the consequence of failure, due to any cause, of the building. Thus, a classification system is established based on the use and size of the building. This classification is called the Risk Category. A combined classification called the Seismic Design Category (SDC) incorporates both the seismic hazard and the Risk Category. The SDC is used throughout the *Provisions* for decisions regarding the application of various specific requirements. The design flow charts in FEMA P-2192-V3 (FEMA, 2021b) illustrate how these classifications are used to control the application of various portions of the *Provisions*.

4. Structural Analysis

The *Provisions* sets forth several procedures for determining the force effect of ground shaking. Analytical procedures are classified by two facets: linear versus nonlinear and dynamic versus equivalent static. The two most fully constrained and frequently used are both linear methods: an equivalent static force procedure and a dynamic modal response spectrum analysis procedure. A third linear method, a full history of dynamic response (previously referred to as a time-history analysis, now referred to as a response-history analysis), and a nonlinear method are also permitted, subject to certain limitations. These methods use real or synthetic ground motions as input but require them to be scaled to the basic response spectrum at the site for the range of periods of interest for the structure in question. Nonlinear analyses are very sensitive to assumptions about structural behavior made in the analysis and to the ground motions used as input, and a peer review is required. A nonlinear static method, also known as a pushover analysis, has been described in prior editions of Part 3 of the *Provisions*, but it is not included in the *Standard*. The *Provisions* also reference ASCE 41, *Seismic Rehabilitation of Existing Buildings*, for the pushover method. The method is instructive for understanding the development of mechanisms, but there is professional disagreement over its utility for validating a structural design.

The two most common linear methods make use of the same design spectrums described previously. The reduction from the elastic spectrum to the design spectrum is accomplished by dividing the elastic spectrum by the coefficient *R*, which ranges from 1-1/4 to 8. Because the design computations are carried out with a design spectrum that is two-thirds the MCE spectrum that means the full reduction from elastic response ranges from 1.9 to 12. The specified elastic spectrum is based on a damping level at 5 percent of critical damping, and a part of the *R* factor accomplishes adjustments in the damping level. Ductility and overstrength make up the larger part of the reduction. The *Provisions* define the total effect of earthquake actions as a combination of the response to horizontal motions (or forces for the equivalent static force method) with response to vertical ground acceleration. The response to vertical ground motion is roughly estimated as a factor (positive or negative) on the dead load force effect. The resulting internal forces are combined with the effects of gravity loads and then compared to the full strength of the members, reduced by a resistance factor, but not by a factor of safety.

With the equivalent static force procedure, the level of the design spectrum is set by determining the appropriate values of basic seismic acceleration, the appropriate soil profile type and the value for *R*. The acceleration for the building is determined from this spectrum by selecting a value for the natural period of vibration. Equations that require only the height and type of structural system are given to approximate the natural period for various building types. (The area and length of shear walls come into play with an optional set of equations.) Calculation of a period based on an analytical model of the structure is encouraged, but limits are placed on the results of such calculations. These limits prevent the use of a very flexible model to obtain a large period and correspondingly low acceleration. Once the overall response acceleration is found, the base shear is obtained by multiplying it by the total effective mass of the building, which is generally the total permanent load.

Once the total lateral force is determined, the equivalent static force procedure specifies how this force is to be distributed along the height of the building. This distribution is based on the results of dynamic studies of relatively uniform buildings and is intended to give an envelope of shear force at each level that is consistent with these studies. This set of forces will produce, particularly in tall buildings, an envelope of gross overturning moment that is larger than many dynamic studies indicate is necessary. In prior editions of the *Provisions*, dynamic analysis was encouraged, and the modal procedure was required for structures with large periods (essentially, this means tall structures) in the higher seismic design categories. Careful nonlinear response history analyses have shown that the reduced strength requirement previously provided for linear modal analysis is not justified, and the *Provisions* now require the same basic strength for both linear methods of analysis.

With one exception, the remainder of the equivalent static force analysis is basically a standard structural analysis. That exception accounts for uncertainties in the location of the center of mass, uncertainties in the strength and stiffness of the structural elements and rotational components in the basic ground shaking. This concept is referred to as horizontal torsion. The *Provisions* requires that the center of force be displaced from the calculated center of mass by an arbitrary amount in either direction (this torsion is referred to as accidental torsion). The twist produced by real and accidental torsion is then compared to a threshold, and if the threshold is exceeded, the accidental torsion must be amplified.

In many respects, the modal analysis procedure is very similar to the equivalent static force procedure. The primary difference is that the natural period and corresponding deflected shape must be known for several of the natural modes of vibration. These are calculated from a mathematical model of the structure. The procedure requires the inclusion of enough modes so that the dynamic response of the analytical model captures at least 90 percent of the mass in the structure that can vibrate. The base shear for each mode is determined from a design spectrum that is essentially the same as that for the static procedure. The distribution of displacements and accelerations (forces) and the resulting story shears, overturning moments and story drifts are determined for each mode directly from the procedure. Total values for subsequent analysis and design are determined by taking the square root of the sum of the squares for each mode. This summation gives a statistical estimate of maximum response when the participation of the various modes is random. If two or more of the modes have very similar periods, more advanced techniques for summing the values are required; these procedures must account for coupling in the response of close modes. The sum of the absolute values for each mode is always conservative.

A lower limit to the base shear determined from the modal analysis procedure is specified based on the static procedure and the approximate periods specified in the static procedure. When this limit is violated, which is common, all results are scaled up in direct proportion. The consideration of horizontal torsion is the same as for the static procedure. Because the equivalent static forces are applied at each floor, the story shears and the overturning moments are separately obtained from the summing procedure, the results are not statically compatible (that is, the moment calculated from the summed floor forces will not match the moment from the summation of moments). Early recognition of this will avoid considerable problems in later analysis and checking.

For structures that are very uniform in a vertical sense, the two procedures give very similar results. The modal analysis method can be better for buildings having unequal story heights, stiffnesses, or masses. Both methods are based on purely elastic behavior, and, thus, neither will give a particularly accurate picture of behavior in an earthquake approaching the design event.

Yielding of one component leads to redistribution of the forces within the structural system; while this may be very significant, none of the linear methods can account for it.

Both common methods require consideration of the stability of the building. The technique is based on elastic amplification of horizontal displacements created by the action of gravity on the displaced masses. A simple factor is calculated, and the amplification is provided for in designing member strengths when the amplification exceeds about 10 percent. The technique is referred to as the P-Delta analysis and is only an approximation of stability at inelastic response levels.

Recent editions of the *Provisions* have incorporated advances in nonlinear response history analysis methods. Such methods of analysis are not required, but they are permitted as an alternate to the linear methods of analysis to validate designs. When used for this purpose, it is possible to demonstrate that buildings will satisfy the intent of the *Provisions*, even though they may:

Have innovative structural systems not otherwise covered by the *Provisions,* ▪ Have a conventional structural system, but do not satisfy some of the empirically based limits, such as maximum height for a shear wall system, ▪ Require demonstration of damage control for vulnerable elements, such as a drift-sensitive cladding system.

When used for such, or similar, purposes the validation analyses must include prediction of response to a suite of ground motions scaled the emulate the MCER response spectrum. Acceptance criteria include limits on strains and deformations of ductile elements and strength of brittle elements. The selections and scaling of ground motions, the analytical modeling of nonlinear response, and the acceptance criteria are all subject to peer review. This method of validation by prediction of performance using sophisticated analysis is often referred to as *performance-based earthquake engineering*, and has led to significant advances in practice, particularly for tall buildings.

5. Nonstructural Elements of Buildings

Severe ground shaking often results in considerable damage to the nonstructural elements of buildings. Damage to nonstructural elements can pose a hazard to life in and of itself, as in the case of heavy partitions or facades, or it can create a hazard if the nonstructural element ceases to function, as in the case of a fire suppression system. Some buildings, such as hospitals and fire stations, need to be functional immediately following an earthquake; therefore, many of their nonstructural elements must remain undamaged.

The *Provisions* treats damage to and from nonstructural elements in three ways. First, indirect protection is provided by an overall limit on structural distortion; the limits specified, however, may not offer enough protection to brittle elements that are rigidly bound by the structure. More restrictive limits are placed upon those Risk Categories for which better performance is desired given the occurrence of strong ground shaking. Second, many components must be anchored for an equivalent static force. Third, the explicit design of some elements (the elements themselves, not just their anchorage) to accommodate specific structural deformations or seismic forces is required.

The dynamic response of the structure provides the dynamic input to the nonstructural component. Some components are rigid with respect to the structure (light weights and small dimensions often lead to fundamental periods of vibration that are very short). The application of the response spectrum concept would indicate that the response history of motion of a building roof to which mechanical equipment is attached looks like a ground motion to the equipment. The response of the component is often amplified above the response of the supporting structure. Response spectra developed from the history of motion of a point on a structure undergoing ground shaking are called floor spectra and are useful in understanding the demands upon nonstructural components.

The *Provisions* simplify the concept greatly. The force for which components are checked depends on:

1. The component mass.
2. An estimate of component acceleration that depends on the structural response acceleration for short period structures, the relative height of the component within the structure and a crude approximation of the flexibility of the component or its anchorage.
3. The available ductility of the component or its anchorage; and 4. The function or importance of the component or the building.

Also included in the *Provisions* is a quantitative measure for the deformation imposed upon nonstructural components. The inertial force demands tend to control the seismic design for isolated or heavy components, whereas the imposed deformations are important for the seismic design for elements that are continuous through multiple levels of a structure or across expansion joints between adjacent structures, such as cladding or piping

6. Quality Assurance

Since strong ground shaking has tended to reveal hidden flaws or *weak links* in buildings, detailed requirements for assuring quality during construction are important. The actively implemented provisions for quality control are contained in the model building codes, such as the *International Building Code* (ICC, 2020) and the material design standards, such as *Seismic Provisions for Structural Steel Buildings*. Loads experienced during construction provide a significant test of the likely performance of ordinary buildings under gravity loads. Tragically, mistakes occasionally will pass this test only to cause failure later, but it is rare. No comparable proof test exists for horizontal loads, and experience has shown that flaws in construction show up in a disappointingly large number of buildings as distress and failure due to earthquakes. This is coupled with the seismic design approach based on excursions into inelastic straining, which is not the case for response to other loads.

The quality assurance provisions require a systematic approach with an emphasis on documentation and communication. The designer who conceives the systems to resist the effects of earthquake forces must identify the elements that are critical for successful performance as well as specify the testing and inspection necessary to confirm that those elements are built to perform as intended. Minimum levels of testing and inspection are specified in the *Provisions* for various types of systems and components.
The quality assurance provisions also require that the contractor and building official be aware of the requirements specified by the designer. Furthermore, those individuals who carry out the necessary inspection and testing must be technically qualified and must communicate the results of their work to all concerned parties. In the final analysis, there is no substitute for a sound design, soundly executed.

7. Resilience-Based Design

Background

In 2018, Congress made it part of NEHRP's purpose to improve community resilience through the development of building codes and standards (Public Law 115-307, 2018). Earthquake resilience is broader than structural design; in fact, resilience is best understood as an attribute of organizations or social units, not of buildings. But seismic design of buildings can contribute to resilience by focusing on the building's post-earthquake *functional recovery* time (EERI, 2019; FEMA-NIST, 2021).

[1] The FEMA-NIST report covers both buildings and infrastructure systems. For clarity, the definitions shown here are edited to address only buildings. The FEMA-NIST report also defines *reoccupancy* and *reoccupancy objective* in a similar way. Reoccupancy is a more basic performance state that precedes functional recovery. Design for reoccupancy is outside the scope of this discussion, but FEMA (2018, Section 5.4.4) provides analytical findings for a selection of model multi-story buildings in terms of the probability of receiving an Unsafe (red) placard after a design earthquake. Except for steel braced frames, the probability is under 15 percent for a Risk Category II design and under two percent for a Risk Category IV design. Thus, for new code-designed buildings, the likelihood of immediate reoccupancy is, as expected, substantially higher than the likelihood of immediate functional recovery, discussed further in Section 2.7.2.2.

The concept of functional recovery discussed in Resource Paper 1 was formalized in a 2021 FEMANIST report with two definitions, one for functional recovery as a performance state, and (consistent with principles of performance-based engineering) one for a design objective that links the performance level with a hazard level and a time-based metric (FEMA-NIST, 2021):[1]

Functional recovery is a post-earthquake performance state in which a building is maintained, or restored, to safely and adequately support the basic intended functions associated with the pre-earthquake use or occupancy.
A functional recovery objective is functional recovery achieved within an acceptable time following a specified earthquake, where the acceptable time might differ for various building uses and occupancies.

Thus, the resilience-based earthquake design of an individual building simply seeks to achieve functional recovery within a specified time after the event. Safety, which is the primary objective of the current *Provisions*, as well as the codes and standards that cite them, remains a floor on the design. Depending on the functional recovery objective, designing for functional recovery might or might not require changes or enhancements relative to the safety-based design.

Current codes and standards do not provide functional recovery design provisions, but the concept of functionality is not entirely new to the *Provisions*. *Provisions* Section 1.1.5 notes that functionality following the design earthquake is the presumed objective for buildings assigned to Risk Category IV, and the 2020 *Provisions* list eight characteristics of a functional building (discussed in Section 2.7.3.2). That said, two important differences between Risk Category IV provisions and functional recovery provisions are:

The element of time. The Risk Category IV provisions expect essentially immediate functionality, just as they expect the building to be safe as soon as the earthquake shaking stops.
By acknowledging that a building might need functional recovery after, say, three days or two weeks, functional recovery provisions can be less conservative than current Risk Category IV provisions.
Consideration of externalities. As shown in Resource Paper 1 (Table 1), functional recovery provisions are likely to be more explicit than Risk Category IV provisions about conditions outside the building footprint, or even outside the scope of traditional design. For example,

functional recovery provisions might include considerations of utility reliability or backup, hazards posed by adjacent buildings, contents damage as it affects “basic intended functions,” or recovery planning as a supplement to design. In this way, functional recovery provisions might be more comprehensive and conservative than current Risk Category IV provisions.

EERI (2019) described four sets of issues that will need to be addressed as a set of functional recovery design provisions are developed:

▪ Definitional. With reference to the FEMA-NIST definition of *functional recovery*, what are the “basic intended functions” of a given building’s use or occupancy, and which physical components are necessary to maintain or restore them? As noted, *Provisions* Section 1.1.5 provides a tentative answer by listing eight characteristics of functionality (discussed in Section 7.3.2), and Resource Paper 1 (Table 1) suggests five categories for functional recovery design provisions: structural, nonstructural, recovery-critical contents, utility service, and preoccupancy and recovery planning.

▪ Policy. With reference to the FEMA-NIST definition of *functional recovery objective*, what is the “acceptable time” for functional recovery, given a building use or occupancy and a prescribed hazard level? Answering these policy questions amounts to selecting, or assigning, functional recovery objectives.

▪ Technical. Given a functional recovery objective, what design provisions will achieve it with appropriate reliability?

▪ Implementation. Should functional recovery design involve new regulations regarding project documentation, licensure, quality assurance, liability, insurance, or legal issues?

The implementation questions are beyond the scope of Section 7. Answers to the definitional questions would be embedded in the technical provisions. Therefore, the balance of this discussion will consider the policy question in Subsection 7.2 and the technical question in Subsections 7.3 and 7.4, considering two contexts: code-based functional recovery design and voluntary functional recovery design.

Functional Recovery Objective

Functional recovery design, like all performance-based design, requires an objective. As defined above, a functional recovery objective requires selection of both a design hazard level and an acceptable functional recovery time.

Building codes set objectives (often implicitly) based on a building’s use and occupancy. For earthquake design, the use and occupancy determine the Risk Category and the Seismic Design Category, which in turn determine the design scope and criteria. As functional recovery provisions are developed for building codes, it is likely that they will also link a functional recovery objective to use and occupancy in some fashion.

The example building (Figure 8) is a six-unit townhouse that would typically be assigned to Risk Category II and Seismic Design Category D. As typical housing, the *International Building Code* (Section 310) would assign it to Occupancy Group R-2, and it would almost certainly have an occupant load under 50.

This discussion assumes R-2 occupancy. But a nearly identical three-story CLT structure could also be used as office suites (Group B) or as a mixed-use building. Considering just residential uses, the same structure with a few modifications might also be used as an assisted living facility (Group I-1) or a nursing home providing medical care (Group I-2). In a larger building, a Group I-2 facility might be assigned to Risk Category III. Beyond the building code's categories, many jurisdictions have policies and programs (supportive housing, rent subsidies, etc.) that might also use a three-story CLT structure. In all these cases, the tenants are vulnerable in the sense that they would likely have difficulty finding alternative housing if forced to relocate, even temporarily, after a damaging earthquake. So, the selection of an appropriate functional recovery objective should consider more than just the basic distinction between residential, institutional, business, or other occupancies. Section 7.2.3 discusses current thinking about appropriate functional recovery times for residential buildings.

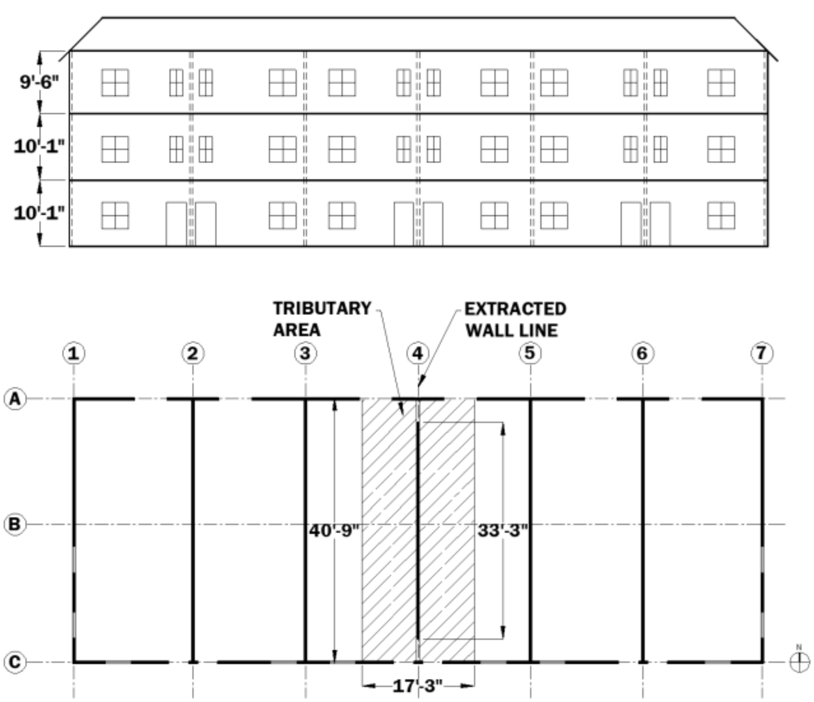

Figure 8. CLT Shear Wall Design Example Building. Top: Elevation. Bottom: Typical Floor Plan Showing Six Townhouse Units

HAZARD LEVEL

Both Resource Paper 1 and the FEMA-NIST report discuss possibilities for an appropriate design hazard level. Arguments can be made for selecting a site-specific hazard different from the hazard currently specified in the *Provisions*, or even a scenario event that better reflects the community resilience perspective.[1] As provisions for functional recovery are developed, an appropriate hazard will be selected through normal consensus processes for developing codes and standards. In the interim, both Resource Paper 1 and the FEMA-NIST report recognize the practicality and convenience of selecting the hazard level for a functional recovery objective to be on par with the *Provisions*' design earthquake. Selecting a much smaller hazard for functional recovery would not add anything to the *Provisions'* current safety-based objective; it would merely restate the assumption that a code designed building will have less damage in a smaller earthquake and more damage in a larger one. Rather, a shift to functional recovery design should mean a heightened interest in functionality, as opposed to just safety, for a similarly rare event.
Therefore, for simplicity and clarity in the absence of a formal consensus, the hazard level selected for this discussion is identical to the *Provisions'* design earthquake.[2]

[1] For further discussion, see Resource Paper 1, Section 2.2, and FEMA-NIST (2021), Chapter 2.

[2] The case studies listed in Table 2-1 of this design example further illustrate how some engineers (and their clients) have selected hazard levels for functional recovery objectives in the absence of a consensus standard.

EXPECTED FUNCTIONAL RECOVERY TIME

Before considering a desired or acceptable functional recovery time, it is useful to consider the functional recovery time achieved by code-compliant Risk Category II designs. Only recently have analytical studies tried to quantify the recovery time of typical buildings. So far, the main finding is that functional recovery time is highly uncertain and can vary substantially between equally code compliant systems.

A FEMA-funded study estimated the repair times for five-story to 13-story code-designed office buildings with five different seismic force-resisting systems, for a range of hazard levels over a range of high seismicity sites (FEMA, 2018, Section 5.4). The repair times estimated by FEMA do not include "additional time required to identify, plan, and permit the work, arrange financing, or hire and mobilize contractors" (FEMA, 2018, Section 5.4.2); the time needed for these activities can often be shortened by advance planning, which Figure 2 of Resource Paper 1 refers to as "reoccupancy and recovery planning." That said, repair time is not the same as functional recovery time. The functional recovery time will be substantially shorter than the repair time if much of the repair can be done while the building is occupied and in use.

For the two concrete systems, the median repair time after a design earthquake is 17 to 21 days. For the three steel systems, it ranges from 15 to 81 days, with two braced frames having the longest repair times. For this discussion, if one assumes that half of the repair time occurs after functional recovery is achieved, the functional repair time may be taken as eight to 40 days (or, to avoid undue precision, one to six weeks).

If designed with Risk Category IV criteria, as would be the case for an emergency operations center, the median repair times reduced to 12 to 15 days for the concrete systems, and 11 to 33 days for the steel systems.2 Risk Category IV facilities tend to have more specialized nonstructural systems and contents, so an even greater portion of the estimated repair time is likely related to their sensitivity. A Risk Category II building designed with Risk Category IV criteria would not have those issues, so to adjust these findings for purposes of this discussion, if one assumes that two-thirds of the Risk Category IV repair time occurs after functional recovery is achieved, the functional repair time may be taken as four to 11 days, or one to two weeks. These median functional recovery times using Risk Category IV criteria for otherwise Risk Category II occupancies would seem to represent the best feasible functional recovery times in the absence of additional recovery planning.

▪ Haselton et al. (2021) used the same FEMA methodology to estimate functional recovery times for wood light-frame (not CLT) residential building types at a high seismicity site in Los Angeles.
For a three- or four-story apartment building, the median functional recovery time was one to six months. The wide range indicates the uncertainty associated with estimates of functional recovery time, which are more complex than those associated with repair. (This study estimated functional recovery time directly, so no adjustment from repair time is needed.)

▪ Furley et al. (2021) estimated reoccupancy and functional recovery times for a two-story office building with CLT walls and supplemental damping devices (that is, different from the Chapter 6 example discussed here). For a spectral acceleration of 1.0 g, typical of a high seismicity area, the median functional recovery time was about 130 days, or four to five months. However, at least half that time was found to be caused by reoccupancy delays related to safety inspections, and beyond that, the actual repair time was driven by nonstructural damage related to the office occupancy and replacement lead times for the damping devices. In addition, the Furley et al. algorithm does not yet account for repairs made while the building is occupied. In a residential building with a plain CLT system and prioritized reoccupancy, the functional recovery time might be substantially shorter.

Thus, for a broad range of newly designed multi-story buildings, one should expect a functional recovery time of at least a few weeks after a design earthquake, and perhaps a few months. An improved design based on current Risk Category IV provisions might reduce the functional recovery time to a few weeks.

That said, a CLT system like the example discussed here is different from any of the systems described above. While there are no studies yet specifically predicting repair time or functional recovery time of typical CLT buildings, there are reasons to think this new system will support faster functional recovery objectives. The system is assigned to a relatively low *R*-factor, and any structural damage is expected to be limited to the ductile steel connectors that are relatively easy to replace, even with the units occupied (Line, 2021). Testing done to quantify the seismic performance factors and to justify the design provisions now in 2020 *Provisions* Section 14.5.2 and SDPWS-21 Appendix B showed "no observable damage in the connections ... and no yielding recorded in the tie-down rods" in a design-level shake table test (van de Lindt et al., 2019a); nail withdrawal of only "a fraction of an inch" after cyclic loading to 2.5% drift (van de Lindt et al., 2021); and reliable nail withdrawal "as expected" when tested to failure (Amini et al., 2016). That said, the Haselton et al. (2021) and Furley et al. (2021) studies cited above also suggest that even with careful selection of the structural system, functional recovery time will be greatly influenced by nonstructural systems and by procedural factors outside the normal scope of building design.

<u>DESIRED OR ACCEPTABLE FUNCTIONAL RECOVERY TIME</u>

Assuming the R-2 occupancy, what is an acceptable functional recovery time? Again, model building codes and standards provide no policy consensus,1 but several jurisdictions and institutions have produced relevant plans that might serve as useful touchstones, if not as policy precedents.

▪ Various "shelter-in-place" and "work-from-home" orders produced during the 2020 pandemic identified a wide range of community services as "essential." While not invoking Risk Category IV design or retrofit provisions, these orders recognized housing and many business types as necessary to community vitality and stability in ways that current building codes do not. They suggested a broader understanding of "substantial economic impact," "mass disruption of day-to-day civilian life," and "substantial hazard to the community" – phrases used to assign risk categories in ASCE/SEI 7-22, Table 1.5-1).

▪ Resilience plans produced by West Coast jurisdictions, organizations, and the federal government have called for building code provisions to explicitly address functional recovery time. Some have focused on specific building uses, but none have yet stated specific functional recovery objectives. (OSSPAC, 2013; White House, 2016; San Francisco, 2016; Los Angeles, 2018)

▪ NIST (2016) calls for local resilience planners to assign different building uses to functional categories and recovery times. Specific assignments should be jurisdiction-specific, but in general, emergency housing, which includes nursing homes and housing for other vulnerable groups, should have "short term" recovery times of at most three days, and other housing should have intermediate recovery times of one to twelve weeks.

▪ The FEMA-NIST report (2021, Table B-1) offers conceptual functional recovery objectives that are generally consistent with NIST (2016). Housing is given as an example of a building use representing "daily necessities" that should have a target functional recovery time of "days to weeks."

▪ SPUR (2009) suggested a set of strawman recovery goals for San Francisco. Accounting for expected performance of the city's existing housing stock, it argued that to meet overall housing goals, new housing should be designed so that 85 percent should be usable within four hours of an M7.2 San Andreas event (somewhat smaller than the design earthquake for most of the city), 95 percent within 24 hours, and 100 percent within 30 days.

▪ For a new senior housing facility, San Francisco set a goal of functional recovery within one day of a 475-year event, intending to eliminate the need for any tenant relocation during repairs (March 2021).

Many of these goals could prove difficult to achieve. They are listed here to indicate the thinking of organizations that have been especially active in the development of earthquake resilience and functional recovery concepts.

In summary, for the townhouse in Chapter 6 CLT shear wall design example:

▪ Separate from any implied objective, a new code-designed multi-story residential building can expect to reach functional recovery within a few months after a design earthquake. If designed as a Risk Category IV facility, the expectation might be to achieve functional recovery within two weeks of a design earthquake. These expectations are based on a limited set of studies with concrete, steel, and wood light frame systems. Testing has suggested that the CLT shear wall system will have limited and highly controlled structural damage in a design earthquake, so the functional recovery time for a CLT building is likely to be shorter.

▪ If a functional recovery objective were specified based on current resilience-based policy suggestions and examples, it might call for functional recovery within at most 30 days of a design earthquake. Current Risk Category II design provisions might not satisfy this objective, but isk Category IV provisions probably will.

▪ If the building might be used as housing for vulnerable tenants without resources to endure 30 days of relocation or limited functionality, the objective might instead call for functional recovery within one to three days of a design earthquake. Even current Risk Category IV design provisions might not satisfy this objective.

Code-based Functional Recovery Design Provisions

As discussed in Table 1 of Resource Paper 1, tentative design provisions to meet different functional recovery objectives might be developed by linking each design strategy already in the *Provisions* to the functional recovery times for which it is needed. Eventually, this mapping will be substantiated by research on the determinants of actual recovery; in the interim, it will be done through consensus processes, with reference to traditional test results.

2020 *Provisions* Section 2.1.5 notes that better performance, as intended for buildings assigned to Risk Category IV, can be achieved by "the increase in the importance factor and more stringent story drift limits, in combination with strict regulation of design, testing, and inspection." As discussed above, FEMA (2018) has shown that selection of the basic seismic force-resisting system (SFRS) can make a significant difference as well. Indeed, the FEMA study suggests that many common systems, as currently codified, cannot reliably achieve a functional recovery time in less than a few days, even with Risk Category IV criteria. Nevertheless, the use of current Risk Category IV criteria will likely continue to be deemed sufficient, by consensus, for the design of any facility for which fast functional recovery is desired, though the current provisions might need to be supplemented with thorough quality assurance and recovery planning.

<u>SEISMIC FORCE-RESISTING SYSTEM</u>

A complete structural design would need to consider the SFRS, diaphragms, foundation, and other non-SFRS walls and framing. This discussion is limited to the CLT shear wall SFRS.
Section 14.5.2 of the 2020 *Provisions* includes design provisions for CLT seismic force-resisting systems. ASCE/SEI 7-22 includes CLT as a new seismic force-resisting system in Table 12.2-1. For CLT design provisions, ASCE/SEI 7-22 references the 2021 *Special Design Provisions for Wind and Seismic* (SDPWS) (AWC, 2020), a material standard referenced here as SDPWS-21. CLT shear wall design, as codified in SDPWS-21, is almost entirely prescriptive. It is based on capacity design principles that ensure yielding primarily in the prescribed steel connections between CLT wall panels, CLT diaphragms, and the foundation (Provisions Section C14.5.2.1 and SDPWS-21 Section C-B.1). Therefore, to the extent that yielding of connectors and fasteners can be limited (without changing the controlling mechanism), the SFRS effect on functional recovery time can be controlled.

Even as a prescriptive design, the new SDPWS-21 provisions for CLT suggest ways, in concept, that a CLT shear wall SFRS might be enhanced to reduce damage and functional recovery time. The discussion below is conceptual only; some elements of the system are specified to ensure a reliable failure of the nailed fasteners, so arbitrary changes to increase the strength or stiffness could affect the failure mode and the overall performance.

▪ Seismic importance factor, *Ie* (ASCE/SEI 7-22 Section 11.5.1). The Seismic Importance Factor is a function of the assigned Risk Category. Nothing in ASCE/SEI 7-22 or the SDPWS-21 prohibits CLT shear walls in Risk Category III or Risk Category IV buildings, so in concept, a Seismic Importance Factor greater than 1.0 could be used with the usual expectation of reducing damage, thereby shortening the structure's effect on functional recovery time. Or, recognizing that resilience and functional recovery are different from safety, recovery-based provisions might introduce a similar, but separate, recovery factor, *Ir*, to do the job. If the intent is to achieve the effect of using Risk Category IV criteria, however, merely increasing the Seismic Importance Factor is not enough, since Risk Category IV criteria also set tighter drift limits and require protection of more nonstructural components.

▪ Height limit (ASCE/SEI 7-22 Table 12.2-1). All else equal, a taller building might be prone to larger forces and deformations, more complicated dynamic response, more damage, and a longer functional recovery time, so a height limit might be a way to control performance. For CLT shear wall systems, however, ASCE/SEI 7-22 Table 12.2-1 sets the same height limit of 65 feet for every Risk Category, indicating that even Risk Category IV performance is achievable up to that height. If there is any benefit to a shorter building, the Chapter 6 design example should already realize it, since its 30-foot height is well under the limit.

▪ Response modification coefficient, *R* (ASCE/SEI 7-22 Table 12.2-1). For CLT systems with panel aspect ratios up to 4, including the Chapter 6 design example, the relatively low *R* value of 3 shows the intent of the 2020 *Provisions* and ASCE/SEI 7-22 to tightly limit even ductile damage.
In more traditional systems, this low value might suggest unreliable or brittle performance. Here, it suggests low damage, which is a key to fast functional recovery. To limit damage even further, one might assign an even lower *R* value, but the same effect is more commonly achieved by assigning a Seismic Importance Factor (or recovery factor) greater than 1.0, as discussed above.

▪ Selection of CLT grade. Grade E1 CLT, as used in the Chapter 6 design example, is one of 14 CLT grades catalogued in the *Standard for Performance-Rated Cross-Laminated Timber (PRG 320)* (APA, 2020) material standard. The properties of the selected grade determine the strength and stiffness of the panel itself. In theory, these can determine the acceptability and expected damage of the design, so different CLT grades might yield different functional recovery times. As shown in the design example, however, the design of this three-story building is controlled by the system's strength, not its stiffness, and that strength is a function of the steel connectors, not the CLT panel (see design example Section 6.5). Therefore, selecting a different CLT grade would probably not affect the functional recovery time in this case.

▪ Classification of CLT walls (2020 *Provisions* Section 14.5.2.2 Items 2, 3, and 4 and SDPWS-21 Section B.2 Items 2, 3, and 4). 2020 *Provisions* Section 14.5.2 and SDPWS-21 Appendix B require the design to account for CLT walls or partitions that might not be needed for overall strength or stiffness and therefore are not considered part of the SFRS. This is to ensure deformation compatibility and to rule out irregularities (2020 *Provisions* Section C.14.5.2.2 and SDPWS-21 Section B.2). If these checks are satisfied, the presence of these walls adds unintended strength and stiffness, potentially reducing damage and functional recovery time. Non-SFRS walls would be difficult to require as part of a design strategy, however, so if additional strength or stiffness is needed, it would be more effective to increase requirements on the SFRS elements, perhaps with a Seismic Importance Factor (or recovery factor) greater than 1.0, as discussed above.

▪ Capacity of prescribed connectors (2020 *Provisions* Sections 14.5.2.3.2, 14.5.2.5, and 14.5.2.6 and SDPWS-21 Sections B.3.2, B.5, and B.6). The strength of a CLT shear wall system is largely a function of the prescribed strength of the prescribed angle connectors at the base of each panel in each story. When these connectors reach their strength in an earthquake, they yield in a controlled way; if the yielding (that is, ductile damage) is enough to require repair, even this reliable and beneficial response can add functional recovery time. A lower *prescribed* capacity for the connectors will require more connectors to be installed for a higher *actual* capacity in the system, which will in turn reduce the expected damage, with a potential reduction in functional recovery time.

Four different parameters directly affect the system strength in each story (represented by the unit shear capacity). Two of these parameters – the connector capacity of 2,605 pounds and the specific gravity factor, *CG* – are derived from tests and are not subject to policy choices. (A different connector could be designed, but that would require new tests; a different wood species could be selected, but that would affect other aspects of the design.) Recovery-based code provisions could, however, adjust the resistance factor, currently prescribed as 0.5 in 2020 *Provisions* Section 14.5.2.6 and SDPWS-21 Section 4.1.1. While there is ample precedent in codes and standards for prescribing different design values for different objectives, in the present case the same effect could be achieved, more transparently, by using a Seismic Importance Factor (or recovery factor) greater than 1.0, as discussed above.

- The modular nature of CLT shear wall design, together with considerations of symmetry and convenience, can sometimes provide additional capacity even without an intentional increase in design requirements. The unintended additional capacity is equivalent to an effective Seismic Importance Factor greater than 1.0.
- Any change that would result in more prescribed connectors along the length of each CLT panel might eventually require connectors on both sides of the wall. Where there is not enough length to stagger them, 2020 *Provisions* Section 14.5.2.3.1 and SDPWS-21

Section B.3.1 require a thicker CLT panel, which will have other effects on both the structural and architectural design.

▪ Deflection calculation and allowable deflection (2020 *Provisions* Section 14.5.2.4, SDPWS-21 Section B.4, and ASCE/SEI 7-22 Table 12.12-1). As noted above, the *Provisions* regard inter story drift as a key metric of performance overall, and high drift is widely understood as an indicator of damage. Tighter drift limits can be expected to reduce damage and shorten functional recovery time. For the CLT shear wall design example, ASCE/SEI 7-22 Table 12.12-1 sets the drift limit at 0.025 times the story height for this Risk Category II residential building because the three-story building is four stories or less and interior walls, partitions, ceilings and exterior wall systems are assumed to have been designed to accommodate story drifts. Design example Section 6.7 shows that the expected building drifts are only about one-third of this limit. Thus, setting tighter drift limits for certain functional recovery objectives would be rational, but at least in this case, even the Risk Category IV limit of 0.015 times the story height is already satisfied and probably would be even if a Seismic Importance Factor (or recovery factor) greater than 1.0 were applied.

▪ Hold-down deformation limit (2020 *Provisions* Section 14.5.2.3.4 Item 2 and SDPWS-21 Section B.3.4 Item 2). CLT shear walls are required to have hold-down devices to resist uplift and overturning. The provisions include a deformation limit of 0.185 inches, derived from criteria for conventional wood framing, intended "to avoid concentration of device elongation in one level" (2020 *Provisions* Section C14.5.2.3 and SDPWS-21 Section C-B.3). In concept, this limit could be tightened to further reduce the potential for disruptive repairs that might delay functional recovery. In design example Section 6.6.1, the estimated elongation is only half of the 0.185-in limit, suggesting that the potential benefit of a tighter limit (if deemed necessary) could be realized with no effect on many typical designs.

▪ Hold-down design force (2020 *Provisions* Section 14.5.2.3.4 Item 3 and SDPWS-21 Section B.3.4 Item 3). Separate from the deformation limit, the hold-down design force must be calculated assuming twice the unit shear capacity of the walls. Since the unit shear capacity is a function of the prescribed connectors, the hold-down design force will increase automatically if the required wall strength is increased as discussed above. Since the purpose of the factor is only to ensure development of the presumed yield mechanism in the connectors (2020 *Provisions* Section C14.5.2.3 and SDPWS-21 Section C-B.3), increasing this factor should have no effect on expected damage or expected functional recovery time.

▪ High aspect ratio panels (2020 *Provisions* Section 14.5.2.3.7 and ASCE/SEI 7-22 Table 12.2-1). In addition to the SFRS used in the Chapter 6 design example, the new provisions allow a CLT shear wall system with a panel aspect ratio of 4. For this system, ASCE/SEI 7-22 Table 12.2-1 allows a somewhat higher R value to reflect the higher displacement capacity of these walls (ASCE/SEI 7-22 Section C12.2-1). While equally safe, a similar building using this system would presumably experience higher drifts and more yielding in the prescribed connectors. Recovery based design provisions might consider prohibiting the high aspect ratio CLT system for buildings with certain functional recovery objectives.

NONSTRUCTURAL SYSTEMS AND CONTENTS

Where structural damage is limited, a building's functional recovery time might be governed by the performance of its nonstructural systems or contents. These are outside the scope of the Chapter 6 design example, but a resilience-based design with a functional recovery objective must consider them.

Except for life safety systems (alarms, exit lighting, fire suppression, etc.) current safety-based design provisions for Risk Category II facilities typically do not seek functionality of nonstructural systems and do not address contents at all. Instead, they require bracing or anchorage only to prevent hazardous materials release and to hold the equipment in place to prevent falling hazards. As with the SFRS criteria, there are no consensus functional recovery design criteria for nonstructural systems and contents, but the *Provisions* do discuss general expectations associated with functionality in Risk Category IV facilities. In general, the design of nonstructural systems for buildings assigned to Risk Category IV must use an importance factor, *Ip*, of 1.5, must brace or anchor smaller components that are exempt for Risk Category II, must ensure backup utility services, and must consider the ruggedness of certain function-critical equipment.

In addition to immediate reoccupancy (which depends on structural performance as well), 2020 *Provisions* Section 1.1.5 lists seven "qualitative characteristics" that define Risk Category IV performance with a design earthquake. The following notes consider these characteristics

▪ Functionality of equipment serving "essential functions." For a non-Risk Category IV building, the "essential functions" are the "basic intended functions" referenced in the FEMA-NIST (2021) definition of functional recovery, given above. For a residential building, they are likely to be the same as those that commonly define habitability in local housing codes – light, ventilation, power, potable water, heat in winter, sanitation and cooking facilities, etc. In some buildings, or for some tenants, elevators and communications systems can be essential as well. These

services are sometimes waived in the immediate aftermath of an earthquake, when basic shelter is the priority, and the duration of the waiver (a policy decision) can help define the functional recovery objective. Current Risk Category II provisions require no design at all for most piping, ducts, floor-mounted equipment, or small suspended equipment. Post-earthquake evaluation of a damaged building for habitability, including reduction in building systems and services, is discussed in detail in FEMA P-2055 (FEMA, 2019).

▪ No damage (or limited damage) to contents serving “essential functions.” Contents generally include any components not constructed with the building but brought in by tenants. For a residential building, “essential” contents might include main kitchen appliances, but in many cases, these are assumed to be part of the building. Tall or suspended furnishings can sometimes pose earthquake risks, but these are not normally essential to the buildings “basic intended function” as housing. Current Risk Category II provisions do not include any design scope for contents.

▪ No damage to non-essential equipment and contents that would “compromise the essential functions.” In a residential building, this category might be understood to include broken glass, fallen ceiling plaster, overturned contents, or other damage that cannot be removed or repaired within the acceptable functional recovery time.

▪ Building envelope “maintains integrity ... to preserve essential functions.” Current code provisions already cover potential damage to glazing, cladding, and roofing components as safety issues. For a residential building, post-earthquake assessment and repair of exterior components such as stucco can often be done from the exterior in ways that do not affect functional recovery.

▪ Nothing more than “minor leakage” in “piping carrying nontoxic substances.”

▪ “Toxic and Highly [sic] toxic substances are not released in a quantity harmful to occupants unless controlled through secondary containment.” Again, functional recovery standards for the full range of building uses will need to parse this general goal. New residential buildings generally do not face risks from release of toxic or hazardous materials.

▪ “Egress is maintained.” Basic safe egress is a prerequisite for reoccupancy, which precedes functional recovery. In a residential building, this objective can usually be met by limiting drifts in the structural design, limiting falling hazards along egress routes, and providing backup power for related mechanical and electrical components. Beyond basic egress, this category might also be understood to include functionality of secondary egress routes and accessibility required in all new construction. As with some habitability issues, strict compliance is sometimes waived in the immediate aftermath of an earthquake.

In considering these nonstructural systems and contents, it is useful to remember that part of the functional recovery objective is the acceptable time to restore function. Even essential equipment or contents damage is acceptable if it can be repaired within the acceptable time. Repair work that can be done while the building is serving its basic intended functions is also acceptable, as buildings routinely undergo planned maintenance, repairs, and alterations without a significant loss of use.

Voluntary Design for Functional Recovery

Resource Paper 1 discusses how the 2020 *Provisions'* current design criteria might be developed to serve functional recovery objectives. The previous section applied that idea, informally, to the new design provisions for CLT shear walls. Until that development occurs through consensus processes, engineers and their clients interested in functional recovery and resilience-based design will implement these concepts voluntarily, usually on a case-by-case basis.

For a project using CLT shear walls as its SFRS, voluntary implementation of resilience-based design can be done by considering the intent and expected outcome of 2020 *Provisions* Section 14.5.2 and SDPWS-21 Appendix B, as well as general performance expectations for structural systems, nonstructural systems, and building contents, as illustrated in the previous section. As noted in Resource Paper 1, consideration should also be given to the availability of utility services and to the potential role of reoccupancy and recovery planning, distinct from building design. For structural design, the engineer might choose to consult academic literature, including test results, for the proposed SFRS; for the CLT shear walls, several of these sources are listed in the References below or are cited by 2020 *Provisions* Section C14.5.2 and the SDPWS-21 Commentary to Appendix B. The engineer might also use a nonlinear analysis procedure to obtain a more complete understanding of likely damage patterns. Procedures and software provided in the FEMA P-58 series (see FEMA, 2018) might also be applied.

Table 1 lists nine recent projects in which engineers and developers voluntarily designed new buildings with organizational resilience or functional recovery objectives in mind. None of the listed projects use CLT shear walls, and only one (March 2021) is a residential building. The examples are offered here only as a resource for engineers interested in how some of their colleagues have implemented concepts of resilience-based design through functional recovery objectives.

Each of the projects had to satisfy appropriate local building codes (which probably referenced design criteria from a prior edition of the *Provisions*), and most ultimately included features not strictly required by those codes. In several cases, the developers or owners already had general performance objectives to supplement the implied objectives of the local building code. In some cases, the engineers and their clients developed objectives and criteria customized to the specific project. The costs of a resilience-based design were typically a concern, and multiple schemes were studied until affordable objectives and designs were selected. Voluntary implementation allows this flexible approach

Table 1. Examples of Voluntary Design for Functional Recovery

Project	Building Use	Functional Recovery Objective or Expectation	Recovery-based Design Features or Criteria
181 Fremont (Almufti et al., 2016)	Office high-rise	Within weeks after design earthquake. (Also, immediate reoccupancy after design earthquake)	Reinforced concrete core, designed using ARUP's REDi criteria
Beaverton, Oregon schools (SEFT, 2015)	Public schools	Risk Category IV performance to serve as post-earthquake shelter	Risk Category IV criteria, backup generator
UCSF Mission Hall (Bade, 2014)	University offices	Operational performance after 84th percentile Hayward event	Enhanced Risk Category II criteria, concrete shear walls
Casa Adelante (Mar, 2021)	Senior housing	Within one day after 475-year event, no tenant relocation	Rocking walls, dampers
85 Bluxome (Moore, 2021)	Office mid-rise	Within "days to weeks" after "major earthquake"	Zero lot lines, SidePlate moment-resisting frame
UCSF Center for Vision Neuroscience (Berkowitz, 2021)	University research	Within 60 days after M7 San Andreas event	I_e = 1.25, 1.5% maximum drift
Oregon Treasury (Zimmerman, 2021)	Government offices	Within zero days after MCE_R	Base isolation, minimized nonstructural systems
Stanford Biomedical Innovations (Lizundia, 2021)	University research	Within 26 days after 475-year event	Modified Risk Category III criteria, element-specific *R* values, I_p = 1.5
Allenby Building (Westermeyer, 2021)	Government offices	Within zero days after 475-year event	Reduced drift limits, amplified demand, post-earthquake recovery plan

References

1. **FEMA P-2192-V1/November 2021 , Volume I: Design Examples , 2020 NEHRP Recommended Seismic Provisions: Design Examples, Training Materials, and DesignFlow Charts , Federal Emergency Management Agency, FEMA .**