Facts for Steel Buildings number 3

Earthquakes and Seismic Design

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INTRODUCTION

Since the introduction of structural steel to building construction, in the late 19th and early 20th centuries, engineers have recognized that steel buildings and structures have performed extremely well compared with structures of other types of construction. One of the earliest and most dramatic examples of the ability of steel structures to withstand a strong earthquake occurred in the great San Francisco earthquake and fire of April 18, 1906. At that time, San Francisco's urban center predominantly consisted of a mixture of light wood-framed and masonry bearing-wall construction. In addition, the city had approximately 30 high-rise buildings constructed with complete vertical load-carrying steel frames and infill masonry walls. The earthquake and fires that followed destroyed almost all of the timber and masonry buildings, but left the steel frame structures. Most of these steel frame structures, which were designed without any consideration of earthquake resistance, were repaired and restored to service, and more than 20 of these structures remain in service today.

The observation of the outstanding performance of steel frame structures in the 1906 San Francisco earthquake led to the requirement in present-day building codes that tall structures must have complete vertical load-carrying frames. Over the years, as California experienced many earthquakes, engineers repeatedly observed that steel frame structures performed in a superior manner relative to other building types. In part, this is why the urban centers of most cities in the western United States, including Los Angeles, San Francisco and Seattle, are composed of steel frame buildings.

By the early 1990s, many engineers in the western United States believed that steel structures were inherently ductile and, as a result, essentially invulnerable to significant earthquake damage. This was reflected in the requirements of building codes of the era. Steel frame structures were permitted to be designed for smaller earthquake forces than buildings of other construction types. Also, relatively few limitations were prescribed on the types of configurations and detailing that could be employed in such structures, relative to the requirements for other types of construction.

The magnitude 6.7 Northridge earthquake that struck the San Fernando Valley, just to the north of Los Angeles, on January 17, 1994, changed this perception. Following the Northridge earthquake, engineers began to discover that a number of steel frame buildings, including both moment frames and braced frames, had experienced significant structural damage, including buckling and fracture of braces in braced frames, and fractures of beam-to-column connections in welded steel moment frames. The damage sustained by moment frame structures was particularly alarming as it became evident that rather than behaving in a ductile manner, these fractures had occurred in a brittle manner. Although no steel frame buildings collapsed in the Northridge earthquake, just one year later, more than 50 steel buildings collapsed in the magnitude 6.8 Kobe, Japan, earthquake of January 17, 1995.

These two events led to massive programs of research into the seismic behavior of steel frame structures, both in Japan and the United States. This research quickly fed into the building codes, and by 1997, the American Institute of Steel Construction (AISC) published a new edition of its *Seismic Provisions for Structural Steel Buildings* (AISC 341) that contained many new requirements affecting the materials, design and construction of steel structures intended to resist strong earthquakes. With the adoption of the *International Building Code* (IBC) throughout the United States, and that code's broad requirements to design structures for seismic resistance, the design criteria contained in AISC 341 has become mandatory in many communities across the United States.

In order to design structures to resist strong earthquakes, it is necessary to have an understanding of structural dynamics and the nonlinear behavior of structures. Structural steel continues to offer several economical and effective means for the design and construction of earthquake-resistant structures. This *Facts for Steel Buildings* presents an overview of the causes of earthquakes, the earthquake effects that damage structures, the structural properties that are effective in minimizing damage, and the organization and intent of seismic design requirements for steel structures in the United States today. More detailed information is available in the references listed at the end of the document.

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SECTION 1
BASIC SEISMOLOGY

1.1 What causes earthquakes?

Earthquakes can be caused by a number of things, including underground explosions, movement of magma within volcanoes, and impacts of large objects with the ground. Earthquakes caused by these types of events generally have very low intensity and rarely cause significant damage. Most damaging earthquakes, however, occur as a result of abrupt movements that occur within the earth's crust.

The earth's crust can be viewed as a thin shell of rock that overlies the planet's molten core. This shell has a number of large cracks in it that effectively divide the crust into a series of very large plates, called tectonic plates. One of these plates underlies much of the North American continent, while another underlies much of the Pacific Ocean. Still others underlie much of Eurasia, and the Indian subcontinent. Figure 1-1 shows a world map that illustrates the layout of these tectonic plates.

Under the influence of gravitational forces, forces induced by the earth's rotation, and forces generated by convection within the earth's molten core, tectonic plates are constantly being pushed against each other, causing stress and strain energy to build up within each plate and along the boundaries between these plates. Over a period of many years, the stresses will accumulate to a point where they exceed the frictional resistance across a plate boundary, or exceed the strength of the rock itself within the interior of a plate. When this occurs, a rapid differential movement of the earth's crust will occur, releasing a portion of the strain energy that has been stored over the years. This strain energy is released in the form of kinetic energy that radiates outward from the zone where the differential movement occurred, causing ground shaking and other earthquake effects.

1.2 Where do earthquakes occur?

Earthquakes can originate anywhere. However, most earthquakes occur along zones of weakness in the earth's crust, which are termed faults, where previous earthquakes have occurred. Faults can often be found along the bases of mountain ranges and hills that were formed by past tectonic activity on these faults. For example, large faults exist along the so-called coastal range of hills in California and along the eastern flank of the Sierra Nevada mountain range. Faults underlie many of the sharp ridges and buttes that can be found in Nevada, Utah and the American Southwest. However, faults also underlie the rolling hills in Texas and Oklahoma and are the geologic features that resulted in the pools of petroleum found in these states.

As the earth's crust changes over the years, through tectonic and geologic activity, so too does the pattern of stress buildup in the crust. Areas of the globe that see extensive earthquake activity in one geologic era may see none in the next. If geologic evidence suggests that movement has

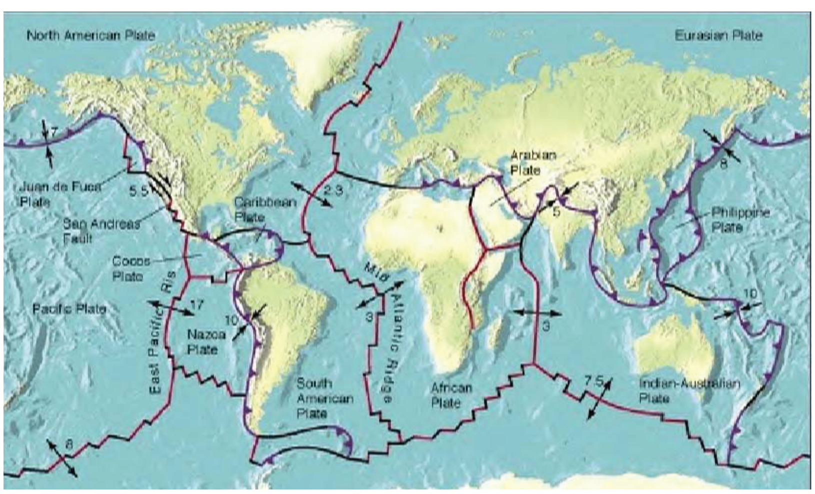


Fig. 1-1. World map illustrating major tectonic plates. (Courtesy of U.S. Geologic Survey)

occurred along a fault in the past 11,000 years, it is termed an active fault. Faults that have not produced earthquakes in this time period are often termed inactive. Most earthquakes occur along active faults, though inactive faults have occasionally been the sources of earthquakes.

In addition to categorization based on their activity, faults are also categorized by the types of slip that occur along them. Faults that primarily have movement consisting of a lateral horizontal displacement along the fault's trace are known as strike slip faults. Faults that have movement consisting of vertical slip along the fault are known as normal faults.

Most of the active faults in the world tend to be located close to the boundaries of the tectonic plates, where stress buildup due to friction along the edges of these plates is rapid and severe. The most active faults are the plate boundaries themselves. The San Andreas Fault, for example, a strike-slip fault that runs along the California coast, is the boundary between the Pacific and North American plates. Further north, this plate boundary underlies the Pacific Ocean just to the west of the North American coast, where it is called the Cascadia Subduction zone.

In subduction zones, one tectonic plate is forced underneath the neighboring plate. In addition to producing some of the world's largest earthquakes, subduction zones also commonly result in volcanic activity. The Cascadia Subduction zone has caused many large earthquakes in Oregon, Washington and British Columbia and also is the origin for the Cascade Range of volcanoes.

To the south, this same plate boundary continues off the coast of Mexico and Central and South America. Another plate boundary extends to the east off the San Andreas system, at the southern tip of Central America; extends into the Caribbean Sea; and then extends northward along the middle of the floor of the Atlantic Ocean.

Because of the extensive buildup of stresses at the plate boundaries and the frequent earthquakes that occur along them, the earth's crust near these boundaries tends to be highly fractured and weak, resulting in many active faults. There are 130 known active faults in the state of California, for example. As distance from the plate boundaries increases, the damage from past earthquakes, the buildup of stress, and the number of active faults all tend to decrease. Nevada and Idaho, for example, tend to have fewer active faults than do the Pacific coast states. Arizona and New Mexico have fewer still. However, there are some regions within the center of the North American continent where major active fault zones exist. These include the Wasatch fault zone that extends through the Salt Lake City region of Utah, and northward towards Yellowstone National Park; the New Madrid fault zone that extends along the Mississippi embayment and northward to the Great Lakes, then northeastward along the St. Lawrence Seaway; and a zone near Charleston, South Carolina. However, earthquakes can and have occurred in many other places, including New England, the Mid-Atlantic States and the Midwest.

Figure 1-2 is a map of the United States, developed by the U.S. Geologic Survey (USGS) that indicates the risk of experiencing damaging earthquakes in various parts of the country. This map indicates areas of the country that have many active faults and which experience frequent earthquakes with a bright red or orange color. Regions that experience occasional earthquakes are shown in yellow to green colors, while areas that seldom experience earthquakes are shown in blue to gray shades. Figure 1-3 is a map of the historic locations of earthquakes in the United States, with magnitudes greater than 4.0.

1.3 How is the severity of an earthquake measured?

There are two basic methods of quantifying the size and severity of an earthquake, respectively termed magnitude and intensity. Magnitude is an objective measure of earthquake size that is used to characterize the amount of energy released by an earthquake event. One of the earliest magnitude scales was developed by C. F. Richter, who measured magnitude on the basis of how much a standard seismic wave measuring instrument deflected, when located a standard distance from the place where an earthquake occurred. Using this system, Richter created a logarithmic magnitude scale ranging from 0, for earthquakes that release negligible energy, to 9 or more, for the largest earthquakes that have ever occurred. Each increase of 1 unit on the Richter magnitude scale represents an increase by approximately 32 times in the amount of energy released. Thus, a magnitude 6 earthquake releases approximately 32 times more energy than a magnitude 5 earthquake, and a magnitude 7 earthquake releases almost 1,000 times more energy than a magnitude 5 earthquake.

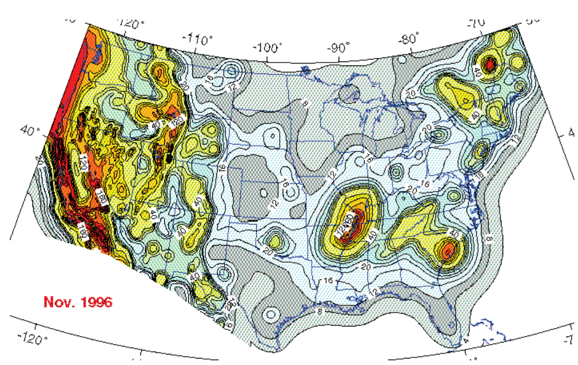


Fig. 1-2. Earthquake risk map for the United States. (Courtesy of U.S. Geologic Survey)

Although the Richter scale is commonly known, and was historically important, it is rarely used by earth scientists to characterize earthquakes. This scale is limited in that when an earthquake occurs, there is almost never a standard instrument present at the standard distance from the earthquake site, which means that approximate conversion formulas must be used to convert the readings of instruments that are available to values that correlate with magnitude. This is one reason why, when following a major earthquake, the news media commonly report different values for the earthquake magnitude: conversion of the readings from different instruments will result in slightly different magnitude estimates. Also, for very large magnitude earthquakes, many seismic measuring instruments tend to dampen out and lose ability to accurately measure the amount of energy released.

For the past 20 years, earth scientists have used the moment magnitude scale to measure earthquake energy. Moment magnitude, which is denoted by the symbol M_w, is a direct calculation of the amount of energy released based on the surface area of the fault that has experienced movement, the amount of slip that has occurred, and the modulus of rigidity of the rock. Since it is impossible to directly measure these quantities, moment magnitude characterizations of earthquakes are also approximate. For small magnitude earthquakes, less than about 7, Richter magnitude and moment magnitude will be similar. For larger earthquakes, moment magnitude tends to be larger, and also more accurate.

While magnitude can be used to describe the amount of energy released by an earthquake, and therefore, its size, it provides little information on the amount of damage the earthquake can cause at a specific site. When an earthquake occurs, the seismic waves radiate outward from the source. As they radiate outward, they decrease in amplitude or attenuate, just as the ripples that form around the place where a stone falls into a pond attenuate with distance. Thus, earthquakes tend to be much more destructive near the source than in a location that is remotely located from it. In some regions, local soil conditions, topography, and other geographic and geologic features can locally focus earthquake energy and amplify it. Thus, it is possible for earthquake shaking to vary in destructive potential for sites that have similar distance from the fault rupture zone. These effects have been commonly measured in the past by intensity scales, which are used to characterize the destructive potential of an earthquake at specific sites.

In the United States, the most commonly used intensity scale is the so-called Modified Mercalli scale. This scale uses Roman numerals that range from I, for earthquake shaking that is not felt, to XII, for earthquake shaking that produces total destruction in a region. Table 1-1 presents the Modified Mercalli scale, as posted on the USGS website. As can be seen in this table, the scale relates to the observed effects of an earthquake at different sites.

Each earthquake will produce different intensities of motions across the affected region. Following an earthquake it is common for the USGS to produce maps of the recorded intensity, and Figure 1-4 is an example of one such map. As can be seen in this figure, earthquake intensity tends to

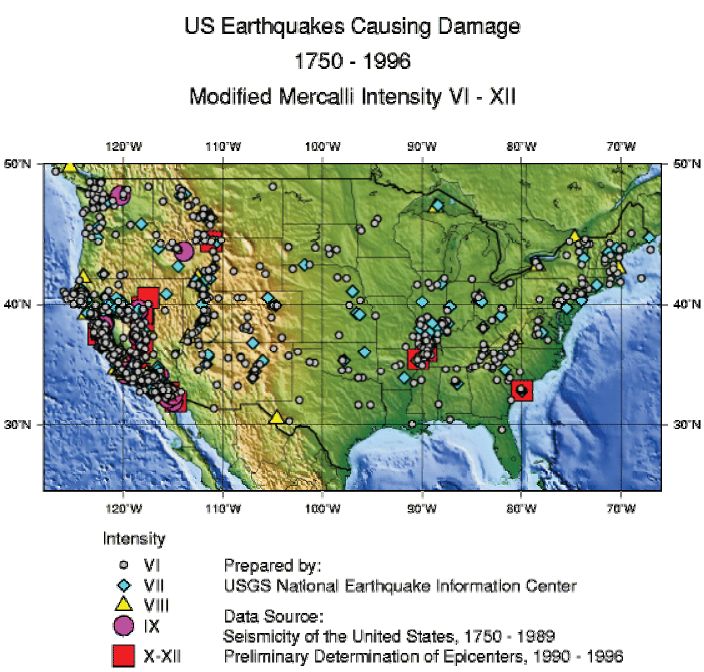


Fig. 1-3. Historic locations of earthquakes in the United States. (Courtesy of U.S. Geologic Survey)

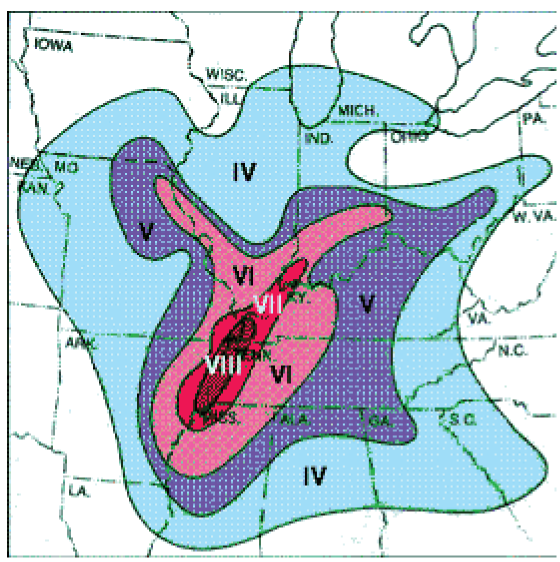


Fig. 1-4. Representative earthquake intensity map. (Courtesy of U.S. Geologic Survey)

Table 1-1 Modified Mercalli Intensity Scale (Courtesy of U.S. Geologic Survey)	
Intensity	**Description**
I	Not felt except by a very few under especially favorable conditions.
II	Felt only by a few persons at rest, especially on upper floors of buildings.
III	Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibrations similar to the passing of a truck. Duration estimated.
IV	Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
V	Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.
VI	Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.
VII	Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.
VIII	Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.
XI	Few, if any (masonry) structures remain standing. Bridges destroyed. Rails bent greatly.
XII	Damage total. Lines of sight and level are distorted. Objects thrown into the air.

Table 1-2 Intensity and Peak Ground Acceleration	
Modified Mercalli Intensity	**Peak Ground Acceleration, *g***
VI	0.05–0.10
VII	0.10–0.20
VIII	0.20–0.30
IX	0.30–0.60
X	> 0.60

diminish with distance from the epicenter, and to be focused in localized pockets, where geologic and topologic conditions amplify the motion.

While intensity is more useful than magnitude as a means of characterizing the destructive potential of an earthquake at a specific site, it is not directly useful for engineering purposes for two reasons. First, it is difficult to predict the potential for intensity at a site until after an earthquake occurs. Second, there is no way to use intensity directly in structural analysis. A number of earth scientists have attempted to correlate intensity with peak ground acceleration, and Table 1-2 presents one such correlation (Trifunac and Brady, 1975). Earthquake ground acceleration values are more useful in structural design, as the amount of force a structure will experience from an earthquake can be calculated from this acceleration.

Note that the primary difference between intensity X and higher levels of intensity are ground failure effects such as liquefaction, lateral spreading, landsliding, etc. Section 1.6 provides more information on these effects.

1.4 How often do earthquakes occur?

Several thousand earthquakes occur throughout the world each year. Most of these earthquakes, however, have very small magnitude, cause no damage, and are not publicized. Very large magnitude earthquakes occur infrequently, with perhaps only one or two moment magnitude 7 earthquakes occurring in any year, and a moment magnitude 8 or larger earthquake occurring only one time every 10 years or so.

When an earthquake occurs on a fault, this releases a portion of the energy that has been stored in the earth's crust and makes it less likely that additional earthquakes will occur on this same fault until additional stress can accumulate. Small-magnitude earthquakes release small amounts of energy and stress, while large-magnitude earthquakes release large amounts of energy. The energy released by a small-magnitude earthquake can be accumulated in a matter of a few years to a few decades. The amount of energy released by a large-magnitude earthquake may take several hundred years—and perhaps several thousand years—to accumulate.

Recurrence relationships are mathematical expressions that indicate the average time, in years, between repeat occurrences of earthquakes of a given magnitude. Recurrence relationships are developed based on the past historic record, either for an entire region, or for particular faults. Earth scientists have developed recurrence relationships for each of the known active faults in the United States.

In Northern California, for example, recurrence relationships suggest that earthquakes with a magnitude between 6.5 and 7, similar to the 1989 Loma Prieta earthquake, will occur approximately one time every 100 years along the San Andreas Fault in the San Francisco region. Magnitude 8 earthquakes like the great 1906 San Francisco event are thought to occur one time every 300 years or so. Great earthquakes along the Cascadia Subduction zone off the coast of Washington State and Oregon are thought to have similar recurrence intervals. The New Madrid fault zone in the area between Memphis and St. Louis is thought to produce very large magnitude earthquakes every 500 years or so, with the last such events having occurred in the winter of 1811–1812. The earthquake source zone near Charleston, South Carolina, is thought to produce large earthquakes one time every 1,000 years.

In areas that are subject to significant earthquake activity, there are often several active faults that can produce destructive earthquakes. Thus, the Los Angeles region has experienced damaging earthquakes every 25 years or so. The San Francisco and Seattle regions have historically experienced highly damaging earthquakes approximately once every 50 years and more frequent, less damaging earthquakes every 25 years. In areas such as these, where damaging earthquakes can be produced by more than one fault, it is common to express the return period of damaging earthquakes in terms of the annual probability of exceedance of ground acceleration as a function of the ground acceleration. These relationships are typically plotted in graphical form, known as hazard curves.

Figure 1-5 is a seismic hazard curve for a site in the City of Berkeley, California, obtained from the USGS web-based ground motion calculation applet. The vertical axis of this plot presents the annual frequency of exceedance of peak ground accelerations of different amounts, shown along the horizontal axis.

The annual frequency of exceedance shown in a hazard curve is equal to the probability that ground shaking of that or greater severity will be experienced in any single year.

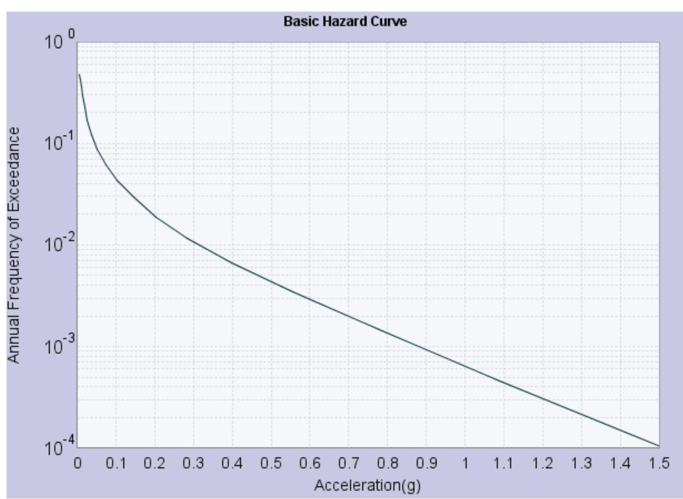


Fig. 1-5. Seismic hazard curve for a site in Berkeley, California.

The inverse of this annual frequency is approximately equal to the average return period in years, for earthquakes of that or greater severity. An annual frequency of exceedance of 0.1, for example, corresponds to shaking that on the average has a 10% chance of occurrence each year. Such shaking would be expected to occur at the site roughly one time every 10 years. An annual frequency of exceedance of 0.01 corresponds to shaking with a 1% chance of occurrence each year, or an average return period of 100 years.

For many years, seismic provisions in U.S. building codes used design ground motion with an annual probability of exceedance of 0.002, or a return period of roughly 500 years. During the deliberations associated with development of the first edition of the IBC, seismologists argued that a return period of 500 years was too short to capture the potential of large earthquakes in the eastern United States such as those that occurred in Charleston, South Carolina, in 1886 or near New Madrid, Missouri, in 1811–1812. In order to capture a repeat of such events, the new code used ground motion with a mean annual frequency of exceedance of 0.0004, or a return period of 2,500 years. Such shaking has a 2% chance of being exceeded in a 50-year period. While this probability of shaking captured very rare, but potentially disastrous events in the eastern United States, it resulted in ground motion that was impractical for design in more seismically active areas, such as coastal California. Therefore, in places of very frequent seismic activity, such as the coast of California, smaller return periods that range between a few hundred to perhaps a thousand years are used, based on a deterministic estimate of the most severe shaking likely to occur in these regions.

1.5 What are the principal effects of earthquakes?

The primary manifestation of an earthquake is the direct permanent displacement of the ground that occurs along the zone of the fault that slips. This displacement can be horizontal, vertical or both and can range from a few centimeters to several meters. Sometimes, the permanent differential ground displacement that occurs along a fault in an earthquake propagates directly to the earth's surface and is visible in the form of a steep fault escarpment, or scarp, for vertical movement (Figure 1-6), or cracks for horizontal ground displacements (Figure 1-7).

The forces produced by such abrupt ground displacements are so large that it becomes impractical to design structures to survive this effect. The best design strategy to avoid damage due to surface fault rupture is to avoid building structures over the traces of known active faults. Fortunately, most buildings are not constructed over these traces and direct fault rupture seldom damages buildings. Surface fault rupture can be very damaging, however, to pipelines, highways, bridges, railroads and other long linear structures that must sometimes cross active faults.

The effect of earthquakes that generally causes the most damage is the violent ground shaking caused by the outward radiation of the energy released by the fault rupture through the rock crust and overlying soils. This ground shaking takes the form of a violent vibration of the ground. Depending on the characteristics of a particular site, its proximity to the zone of fault rupture, and the type of rupture and its magnitude, the vibration can have broad frequency content with destructive shaking having frequencies of 0.2 Hz to 100 Hz.

Fig. 1-6. Fault scarp created by the 1954 Dixie Valley earthquake in the Nevada desert. (Photo by K.V. Steinbrugge)

Fig. 1-7. Offset of highway centerline, 1988 Spitak, Armenia, earthquake. (Photo by P. Yanev)

Ground shaking causes more than 90% of the earthquake damage to the built environment, and is the primary earthquake hazard addressed by the building codes.

In addition to damaging structures, violent ground shaking can also cause instability of the ground. The most common shaking-induced ground instability is landsliding, which is often caused by earthquakes on steeply sloping sites. Earthquake-induced landslides can be very large and have been known to destroy entire residential subdivisions and downtown districts.

Another ground instability caused by earthquakes is soil liquefaction. When strong ground shaking occurs in loose granular soils, including silts and sands, this tends to densify the material. If the soil is saturated, as it densifies the particles move downward, forcing the ground water upward. Very high ground water pressures can result, causing temporary geysers to erupt, ejecting water and soil from the ground. As this material is ejected, the ground can experience large differential settlement. In addition, while liquefaction is occurring, a quick condition can develop in the soils, with a temporary loss of effective stress in the soils and loss of bearing capacity. When this occurs, structures supported on the soils can experience extreme settlements. One such case is illustrated in Figure 1-8.

If liquefaction occurs on a sloping site or adjacent to a steep cut, such as often exists along rivers, the liquefied soils can begin to flow downward and outward, causing large, nonuniform, permanent vertical and horizontal displacement of the ground surface. As illustrated in Figure 1-9, this phenomenon is known as lateral spreading. Liquefaction has been a frequent source of damage for bridges and ports.

1.6 How do earthquakes affect buildings?

If fault rupture, landsliding, liquefaction, or lateral spreading occurs at a building site, the resulting permanent ground deformations can tear a structure apart. Consequently, it is very difficult to design structures to resist these effects.

Some foundation types are better able to resist these permanent ground deformations than others and provide some protection for structures. For example, the use of pile foundations, with the piles extending beneath the anticipated zone of soil liquefaction can be an effective method for mitigating the effects of that hazard. The use of heavily reinforced mats can also be effective in resisting moderate ground deformation due to fault rupture or lateral spreading.

Most earthquake-induced building damage, however, is a result of building response to ground shaking. When the ground shakes at a building site, the building's foundations will vibrate in a similar manner to the surrounding ground. Because all structures have mass—and, therefore, inertia—as well as some flexibility, the structure will lag behind somewhat when the ground and foundations begin to move. That is, the base of the structure will displace, both laterally and vertically, relative to the elevated floors and roof. Then, because the structure has stiffness, the relative displacement induced in the structure will produce forces, which then will produce further deformation of the structure. This process will repeat throughout the duration of the earthquake, with floors and roof moving relative to the ground and to each other. Once the ground shaking stops, damping in the structure will eventually dissipate the energy delivered to it by the earthquake, and the structure will come to rest.

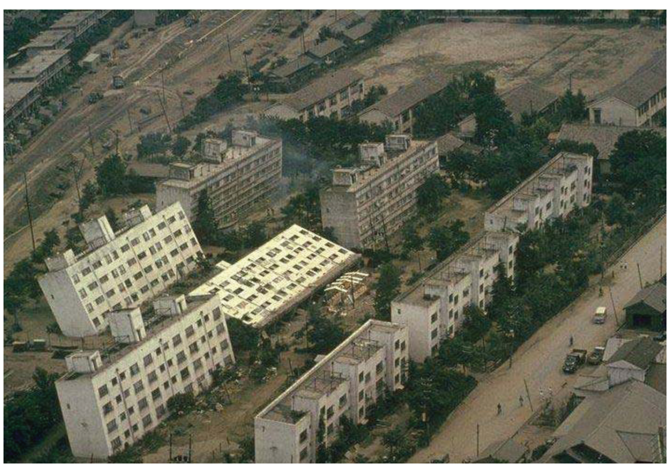


Fig. 1-8. Settlement in apartment buildings due to soil liquefaction, 1964 Nigata, Japan, earthquake. (Photo courtesy of University of Washington)

Fig. 1-9. Lateral spreading damage to highway pavement near Yellowstone Park, 1959 Hegben Lake earthquake. (Photo courtesy of U.S. Geologic Survey)

The amount of force and deformation induced in a structure by an earthquake is a function of the amplitude and frequency content of the ground shaking, the structure's dynamic properties, and its strength. Every structure has certain unique natural modes of vibration, each characterized by a deformed shape and frequency. These natural modes are functions of the structures mass and stiffness distribution.

If a structure is displaced into a deformed shape that matches one of its natural modes, and then released, it will vibrate back and forth in this deformed shape at the modal frequency until the motion is damped out. Earthquakes, having broad frequency content, will tend to excite structures in each of their natural modes so that the structure experiences vibration in several deformed shapes, simultaneously. If a particular ground motion has strong energy content at a frequency that is similar to one or more of the structure's natural modes, the structure will develop resonance and vibrate strongly in that mode.

Since the natural modes of each building are unique, one earthquake will tend to affect each building differently, with some buildings experiencing strong response in some modes of vibration and other buildings experiencing strong response in other modes. Furthermore, as structures are damaged by strong shaking, their stiffness changes, as does their modal properties. Sometimes this stiffness change is beneficial and allows a building to detune itself from the strongest effects of shaking. Other times, this change in modal properties results in the earthquake delivering more energy to the structure causing still more damage. These effects are made more complex by the fact that even in a single earthquake the character of the ground shaking experienced at each site tends to be somewhat different. Thus, it is not uncommon to see similar buildings on nearby sites, affected very differently by a single earthquake.

Buildings experience structural damage when the deformations and forces induced in the structure by its response to the ground shaking exceed the strength of some elements. Brittle elements in such a structure will tend to break and lose strength. Examples of brittle elements include unreinforced masonry walls that crack when overstressed in shear and unconfined concrete elements that crush under compressive overloads. Ductile elements are able to deform beyond their elastic strength limit and continue to carry load. Examples of ductile elements include tension braces and adequately braced beams in moment frames.

As structural elements are damaged in an earthquake, the structure will become both weaker and more flexible, and as a result, the lateral deformations can become very large. If lateral deformation becomes too large, the structure can develop P-Δ instability and collapse. Local collapse can occur when gravity load-carrying elements, like beams or columns, are damaged so severely that they can no longer support the weight of the structure. Nonstructural elements, including cladding, ceiling systems, mechanical equipment and piping, can also be damaged by earthquake shaking.

Even in regions of very high seismic risk, like coastal California, severe earthquakes occur infrequently. Most buildings will never experience an earthquake strong enough to cause extensive damage. Therefore, for economic reasons, building codes have adopted a design philosophy that permits the design of buildings such that they would be damaged by the infrequent severe earthquakes that may affect them, while attempting to require sufficient resistance to prevent collapse and gross endangerment of life safety. For those few buildings that house important functions that are essential to post-earthquake recovery, including hospitals, fire stations, emergency communications centers and similar structures, building codes adopt more conservative criteria, which are intended to minimize the risk that the buildings would be so severely damaged they could not be used for their intended function after the earthquake.

SECTION 2
BASIC EARTHQUAKE ENGINEERING

2.1 What are a structure's important dynamic properties?

The amount and way that a structure deforms in an earthquake, termed its response, are a function of the strength and dynamic properties of the ground shaking, as well as those of the structure itself. The principal dynamic properties of importance to structural earthquake response are the structure's modal properties and its damping.

The simplest type of structure is the so-called single degree of freedom (SDOF) structure. An SDOF structure has all of its mass concentrated at a single location, and this mass is constrained to move in only one plane. A classical model of an SDOF structure consists of a single concentrated mass, M, on top of a cantilevered column. Figure 2-1 represents such a model. If, as shown in the figure, a force, F, is statically applied to the mass, the column will deform laterally, allowing the mass to displace in the direction of the applied force. If the column has stiffness, K, it will deflect to a displacement x, given in Equation 2-1.

$$x = F/K \tag{2-1}$$

If the mass is maintained in equilibrium, the column will experience a shear force equal and opposite to the applied external force, F. If this force is suddenly removed, the structure will continue to exert a force, $-F$, on the mass, which will cause the mass to accelerate back toward its at-rest position. As the mass moves back toward the center position, the force in the column will decrease, until as the mass moves to the initial at-rest position the column will have no shear force. However, the mass, now having inertia, will continue to move through and away from the initial at-rest position, in a direction opposite to the original applied force. In the process, the column will begin to exert shear forces on the mass in opposition to the direction of motion, and slow the mass until eventually, it comes to rest at position $-x$. Again the force in the column will accelerate the mass back toward the initial at-rest position, causing a back-and-forth vibration, with maximum amplitudes $+x$ and $-x$ at a unique natural frequency given by Equation 2-2.

$$f = \frac{1}{2\pi}\sqrt{\frac{K}{M}} = \frac{1}{2\pi}\sqrt{\frac{Kg}{W}} \tag{2-2}$$

In this equation, W is the weight of mass M, g is the acceleration due to gravity, and the frequency, f, has units of cycles/second.

In earthquake engineering, it is common to use the inverse of the frequency, termed the period, which is the time, in seconds, it would take the structure to undergo one complete cycle of free vibration from $+x$ to $-x$ to $+x$. This period, which is usually represented by the symbol T, is given by the Equation 2-3.

$$T = 2\pi\sqrt{\frac{W}{Kg}} \tag{2-3}$$

Real buildings will have at least three significant dynamic degrees of freedom at each level, consisting of a horizontal translational degree of freedom in each of two orthogonal directions and a rotational degree of freedom about the vertical axis. Single-story structures that have orthogonal seismic load resisting systems and coincident center of mass and stiffness can be accurately represented as a series of two SDOF models, each representing the building's behavior in one of the directions of lateral force resistance. In such models, the weight of the roof, roof-mounted equipment, and suspended ceilings, as well as the weight of the upper half of walls are considered to be lumped at the center of mass of the roof. The stiffness is calculated separately for each of the two orthogonal directions, and Equation 2-3 can be used to determine the structure's period in each direction of response.

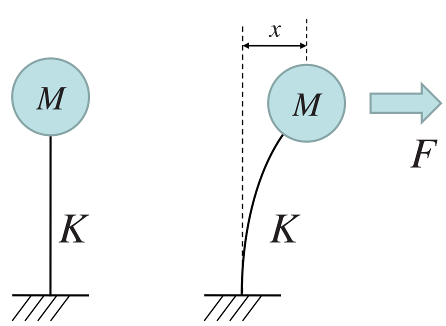


Fig. 2-1. Mathematical model of SDOF structure.

Multi-story structures must be treated as multi-degree of freedom (MDOF) structures. As with single-story structures, if the centers of stiffness and mass at each level are coincident and align vertically from story to story, torsional degrees of freedom can generally be neglected. The earthquake response of such structures can be calculated using a stick model with the mass in each story lumped at a single point, and the stiffness of the seismic load resisting system in each story can be represented by a single translational spring, as illustrated in Figure 2-2 for a three-story structure.

MDOF structures will have one natural mode of vibration, i, for each degree of freedom, j. Each mode of vibration will have a unique period, T_i, and a unique deformed shape, ϕ_i, at which it will undergo free vibration. These deformed shapes are called mode shapes. Figure 2-3 illustrates the three mode shapes for the three-story structure shown in Figure 2-2.

The displaced shapes for each mode are commonly assembled into a modal shape vector, denoted by the symbol ϕ_i, where i is the mode number. A value of $i = 1$ typically is assigned to the mode that has the lowest natural frequency (and longest period). The entries in this vector are the relative deformed shape displacements, $\phi_{i,j}$, where i is the mode number and j is the degree of freedom number. The modal shape vector, ϕ_i, can be normalized to any value; however, it is common practice to normalize the shape vectors such that the quantity $\phi_i^T M \phi_i$ has a value of unity. M is the structure's mass matrix, which is a diagonal matrix with entries (m_1, m_2, …, m_n), where each quantity mj is the mass at degree of freedom j.

In any natural mode shape for an MDOF structure some of the masses move more than others. As a result, only a portion of the structure's mass is effectively mobilized during vibration in a particular mode. The effective or modal mass M_i for mode i is given by Equation 2-4.

$$M_i = \frac{\left(\sum m_j \phi_{i,j}\right)^2}{\sum m_j \phi_{i,j}^{\,2}} \tag{2-4}$$

In this equation, mj is the lumped mass at degree of freedom j and $\phi_{i,j}$ is the relative deformed shape displacement for mode i at degree of freedom j. The sum of the modal masses for all of a structure's modes is equal to the structure's total mass.

A convenient way to analyze the earthquake response of a structure is to analyze the structure as a series of SDOF structures, each having the modal mass, M_i, and period, T_i, of one of the structure's natural modes. Generally, this form of analysis is considered to be sufficiently accurate if enough modes have been evaluated such that the sum of the modal masses, M_i, for each of the modes considered is equal to at least 90% of the structure's total mass. When this type of analysis is performed, it is necessary to transform the results obtained from each mode of analysis by a participation factor, α, which is given by Equation 2-5.

$$\alpha = \frac{\phi_{i,n} \sum m_j}{\sum \phi_{i,j} m_j} \tag{2-5}$$

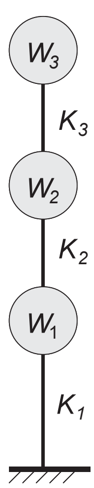


Fig. 2-2. Multi-degree of freedom model representing a three-story structure.

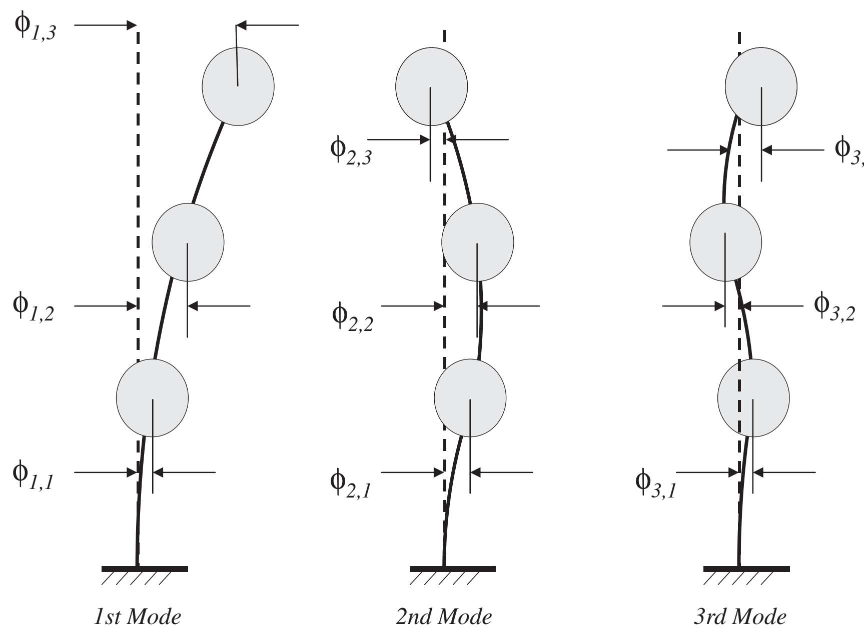


Fig. 2-3. Representative modal shapes for a structure with three degrees of freedom.

In this equation, $\phi_{i,n}$ is the largest relative deformed shape displacement for mode i and the other quantities are as previously defined.

A final dynamic property of importance in earthquake analysis is the structure's effective damping. Damping is a form of energy dissipation that is inherent in all structures. In classic dynamic theory, damping is viewed as a viscous form of energy dissipation, proportional to the velocity of the structure at any instant of time. In real structures, damping is a function of viscous energy losses, friction and energy dissipated by inelastic structural behavior, which is also termed hysteresis. Sources of damping in buildings include energy dissipated by nonstructural elements, frictional dissipation of energy at bolted connections and yielding of structural members.

It is common to express a structure's damping in terms of the fraction of critical damping that is present. Critical damping is the minimum amount of damping that is required to bring a structure that is displaced from its position and then released to rest, at its original un-displaced position, without vibration. For SDOF structures, the critical damping, C_c, is given by Equation 2-6.

$$C_c = \sqrt{4KM} = \sqrt{\frac{4KW}{g}} \tag{2-6}$$

When performing linear or elastic analysis of a building's response to earthquake shaking, it is common to assume that it inherently has 5% of the critical damping. In actuality, most steel structures have somewhat less damping than this when behaving elastically. The amount of damping that can actually be mobilized depends on many factors, including the amplitude of vibration and the amount of damage, if any, that occurs.

2.2 What is response history analysis?

Response history analysis, which is sometimes called time history analysis, is a method of calculating the response of a structure to a specific earthquake ground motion through numerical integration of the equation of motion (Equation 2-7).

$$M\ddot{x}(t) + C\dot{x}(t) + Kx(t) = -M\ddot{x}_g(t) \tag{2-7}$$

For SDOF structures, M is the mass, C is the damping, K is the stiffness and $x(t)$, $\dot{x}(t)$ and $\ddot{x}(t)$ are, respectively, the structure's displacement, velocity and acceleration relative to the ground at an instant of time t. The quantity $x_g(t)$ is the acceleration of the ground at an instant of time t.

In order to perform response history analysis, it is necessary to have a digitized ground motion acceleration record. In linear response history analysis, the stiffness of the structure, K, is assumed to be independent of the prior displacement history. In nonlinear response history analysis, the structure's stiffness at an instant of time t, is dependent on the displacement history up to that point in time and varies to account for yielding, buckling and other behaviors that may have occurred earlier in the structure's response.

Response history analysis is useful because it allows solution of the deflected shape and force state of the structure at each instant of time during the earthquake. Since each earthquake record has different characteristics, the results obtained from response history analysis are valid only for the particular earthquake record analyzed. Therefore, when performing response history analysis to determine forces and displacements for use in design, it is necessary to run a suite of analyses, each using different ground motion records as input. Present building codes require a minimum of three records. If three records are used, the maximum forces and displacements obtained from any of the analyses must be used for design purposes. If seven or more records are used, the code permits use of the mean forces and displacements obtained from the suite of analyses.

In design practice, linear response history analysis is seldom used. This is because for design purposes, one is usually interested only in the maximum values of the response quantities (forces and displacements) and these quantities can more easily be approximated by an alternative form of analysis known as response spectrum analysis (see Section 2.4). Nonlinear response history analysis is increasingly used in design projects. It is an essential part of the design of structures using seismic isolation or energy dissipation technologies, and it can be quite useful in performance-based design approaches.

2.3 What is an acceleration response spectrum?

An acceleration response spectrum is a plot of the maximum acceleration $x(T)$ that SDOF structures having different periods, T, would experience when subjected to a specific earthquake ground motion. This plot is constructed by performing response history analyses for a series of structures, each having a different period, T, obtaining the maximum acceleration of each structure from the analysis, and plotting this as a function of T. Linear acceleration response spectra are most common, and are obtained by performing linear response history analysis. Figure 2-4 shows a typical linear acceleration response spectrum obtained from a record from the 1940 Imperial Valley earthquake.

Although the response spectra obtained from each earthquake record will be different, spectra obtained from earthquakes having similar magnitudes on sites with similar characteristics tend to have common characteristics. This has permitted the building codes to adopt standard response spectra that incorporate these characteristics, and which envelope spectra that would be anticipated at a building site during a design earthquake. The response spectra contained in the building code are called smoothed design spectra because the peaks and valleys that are common in the spectrum obtained from any single record are averaged out to form smooth functional forms that generally envelope the real spectra.

2.4 What is response spectrum analysis?

Response spectrum analysis is a means of using acceleration response spectra to determine the maximum forces and displacements in a structure that remains elastic when it responds to ground shaking. For SDOF structures, the maximum elastic structural displacement is given by Equation 2-8.

$$\Delta = \frac{T^2}{4\pi^2} S_a(T) \tag{2-8}$$

In this equation, T_i is the structure's period and $S_a(T)$ is the spectral acceleration obtained from the response spectrum plot at period T. The maximum force demand on the structure is given by Equation 2-9.

$$F = \frac{W}{g} S_a\left(T\right) = K\Delta \tag{2-9}$$

For MDOF structures the response of the structure can be determined by calculating and combining the response quantities for a series of SDOF structures having the same period and mass as each of the structure's modes. For mode i the maximum inertial force produced in the structure by the earthquake, which is also termed the modal base shear, V_i, is given by Equation 2-10.

$$V_i = M_i S_a\left(T_i\right) \tag{2-10}$$

In this equation, M_i is the modal mass for mode i and $S_a(T_i)$ is the spectral acceleration obtained from the response spectrum at natural period T_i.

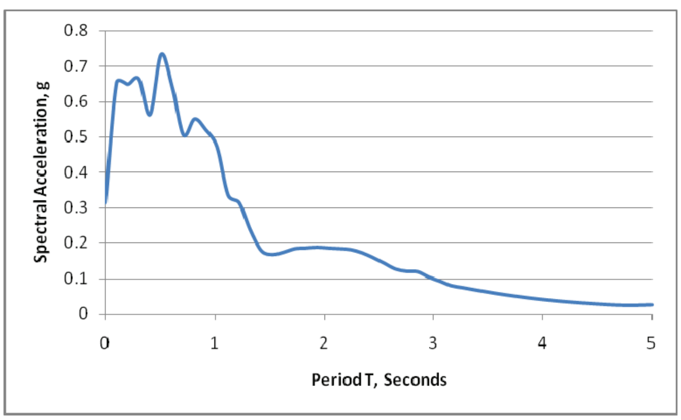


Fig. 2-4. Linear acceleration response spectrum, 1940 El Centro, 180° component, 5% damping.

The inertial force at each degree of freedom j for mode i is given by Equation 2-11.

$$F_{i,j} = \alpha_i \phi_{i,j} m_j S_{ai} \tag{2-11}$$

In this equation, α_i is the modal participation factor for mode i, m_j is the mass at degree of freedom j, S_{ai} is the spectral response acceleration for mode i, and $\phi_{i,j}$ is the modal displacement of degree of freedom j in mode i normalized, as previously described, which can be determined by performing a static analysis of the structure for a load case consisting of the application of the inertial forces, $F_{i,j}$.

The results of the analyses conducted for the various modes must be combined in order to obtain an estimate of the structure's actual behavior. Since it is unlikely that peak structural response in all modes will occur simultaneously, statistical combination rules are used to combine the modal results in a manner that more realistically assesses the probable combined effect of these modes. One such combination method takes the combined value as the square root of the sum of the squares (SRSS) of the peak response quantities in each mode.

When several modes have similar periods, the SRSS method does not adequately account for modal interaction. In this case, the complete quadratic combination (CQC) technique is more appropriate. While detailed discussion of the basis and means of application of these techniques is beyond the scope of this document, many textbooks on earthquake analysis provide discussion of these methods, and most structural analysis software used in design offices today provides the capability to perform these computations automatically.

For SDOF structures, response spectrum analysis gives exact results, as long as the response spectrum that is used to represent the loading accurately represents the ground motion. As noted in Section 2.3, however, the response spectra contained in building codes only approximate the ground motion from real earthquakes, and therefore, analysis using these spectra will be approximate. For MDOF structures, response spectrum analysis is always approximate because the way that the peak displacements and forces from the various modes are combined does not accurately represent the way these quantities will actually combine in a real structure subjected to real shaking. Although the results of response spectrum analysis are approximate, it is universally accepted as a basis for earthquake-resistant design, when properly performed.

2.5 What is inelastic response?

Inelastic response occurs when the amplitude of earthquake shaking is strong enough to cause forces in a structure that exceed the strength of any of the structure's elements or connections. When this occurs, the structure may experience a variety of behaviors. If the elements that are strained beyond their elastic strength limit are brittle, they will tend to break and lose the ability to resist any further load. This type of behavior is typified by a steel tension member that is stretched such that the force in the brace exceeds the ultimate strength of its end connections or by an unreinforced concrete element that is strained beyond its cracking strength. If the element is ductile, it may exhibit plastic behavior, being able to maintain its yield strength as it is strained beyond its elastic limit. This type of behavior is typified by properly braced, compact section beams in moment frames; by the cores of buckling-restrained braces; and by the shear links in eccentrically braced frames. Even elements that are ductile and capable of exhibiting significant post-yielding deformation without failure will eventually break and lose load-carrying capacity due to low-cycle fatigue if plastically strained over a number of cycles.

Modern structural analysis software provides the capability to analyze structures at deformation levels that exceed their elastic limit. In order to do this, these programs require input on the hysteretic (nonlinear force vs. deformation) properties of the deforming elements.

Figure 2-5 shows a hysteretic plot for a theoretical element that has elastic-perfectly plastic properties. In this behavior, the structure loads and unloads at an elastic stiffness, K. When it is loaded to its yield strength, F_y, either in tension or compression, it will continue to deform while maintaining constant strength, until it reaches an ultimate deformation, δ_u, at which point it will break and lose both stiffness and strength.

Few elements in real structures are capable of exhibiting true elastic-perfectly plastic behavior. However, many elements in steel structures are capable of exhibiting a form of this behavior known as elastic-plastic strain hardening behavior, which is illustrated in Figure 2-6. In this behavior, the element loads and unloads at a constant elastic stiffness until it reaches a yield deformation, at which point it continues to load at a reduced, post-elastic stiffness. Typical post-elastic stiffness of steel elements varies from between 5% to 20% of the initial elastic stiffness. With each cycle of loading beyond the prior yield point, the element strain hardens, forming a new higher yield point and yield strain. As with elastic-perfectly plastic elements, the elastic-strain hardening element will lose stiffness and strength if it is loaded to sufficiently large strains. Steel elements that exhibit this behavior include buckling-restrained braces, shear links in eccentrically braced frames, and properly braced compact-section beams in moment frames. Figure 2-7 shows actual hysteretic data obtained from a test of a buckling-restrained brace (see Section 5.3). Figure 2-8 shows a similar plot for a moment-resisting beam-to-column joint using the Welded Unreinforced Flange moment connection.

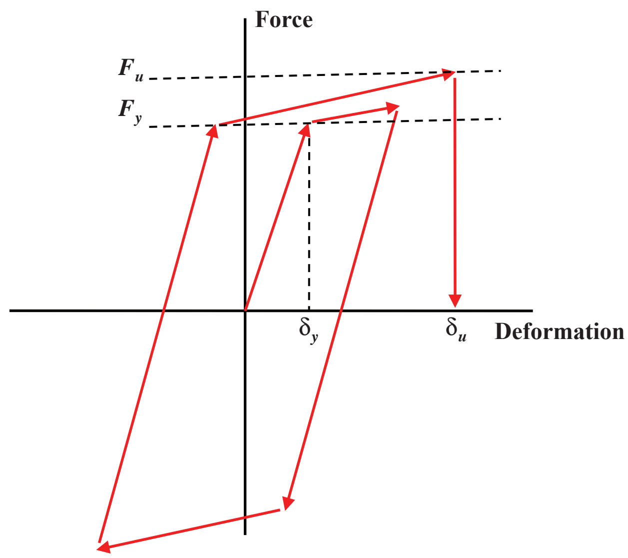


Fig. 2-6. Elastic-plastic strain hardening behavior.

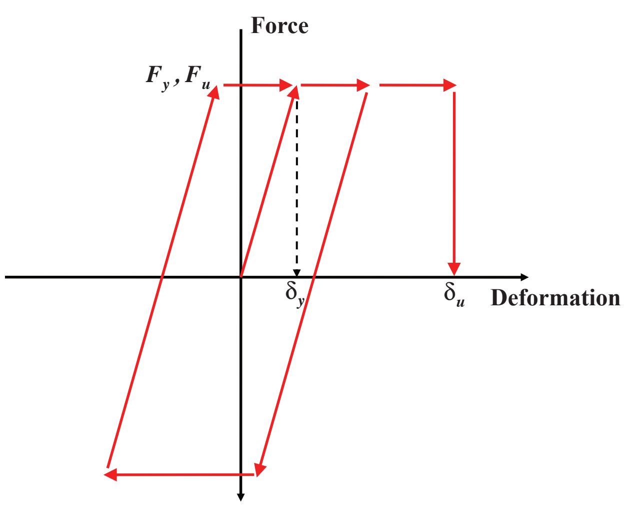


Fig. 2-5. Elastic-perfectly plastic hysteretic behavior.

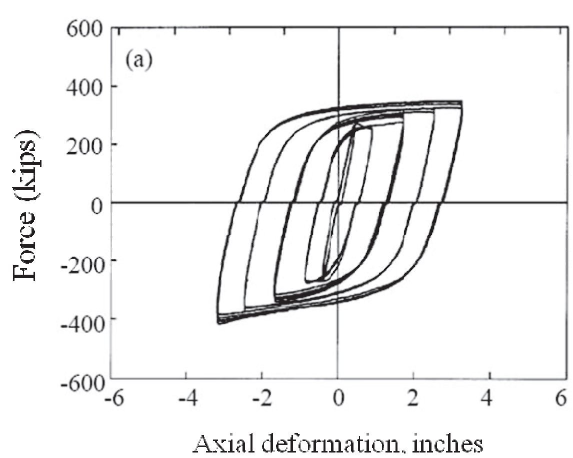


Fig. 2-7. Hysteretic data from test of buckling-restrained brace.

Some structural elements exhibit ductile post-elastic behavior that includes strength degradation after yielding. Elements that exhibit this behavior include beams that are inelastically strained in flexure but that are noncompact and exhibit local flange buckling, as well as beams that exhibit lateral torsional buckling. The strength degradation that occurs in such framing is sometimes considered a special case of elastic-plastic strain hardening behavior in which the strain hardening slope is negative. This is sometimes termed elastic-plastic strain degrading behavior. Figure 2-9 illustrates such behavior in a reduced beam section (RBS) moment connection without adequate bracing of the beam flange at the plastic hinge. Similar cyclic degradation in strength will also occur when an element undergoes large-amplitude buckling, either globally or locally.

Pinching is a type of behavior in which the unloading stiffness of the structure is significantly less than the initial elastic stiffness. Pinching behavior occurs in wood wall panel systems due to slip in the nailing of the sheathing to the framing and in reinforced concrete elements due to opening and closing of cracks in the concrete and loss of bond of the reinforcing steel to the concrete. Figure 2-10 shows a hysteretic curve for a reinforced concrete column, illustrating this behavior.

For many years engineers believed that hysteretic pinching was an undesirable characteristic that would lead to larger structural displacements during inelastic response. However, recent research indicates that hysteretic pinching without strength degradation does not produce undesirable response and, in some cases, can produce less structural deformation than elastic-perfectly plastic behavior. Pinching coupled with significant strength degradation, however, is known to produce very large inelastic response in structures and can lead to collapse. This behavior is typical of buckling in braces as illustrated in Figure 2-11.

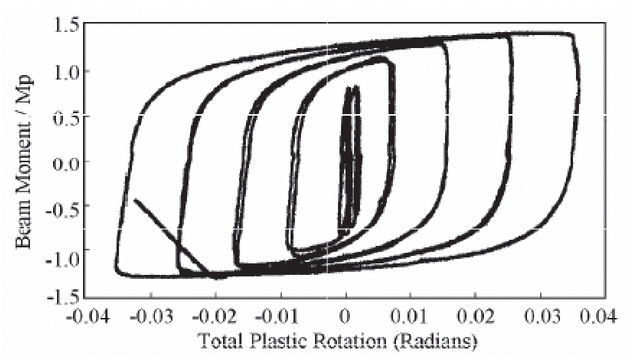


Fig. 2-8. Hysteretic data from test of Welded Unreinforced Flange beam-to-column moment connection.

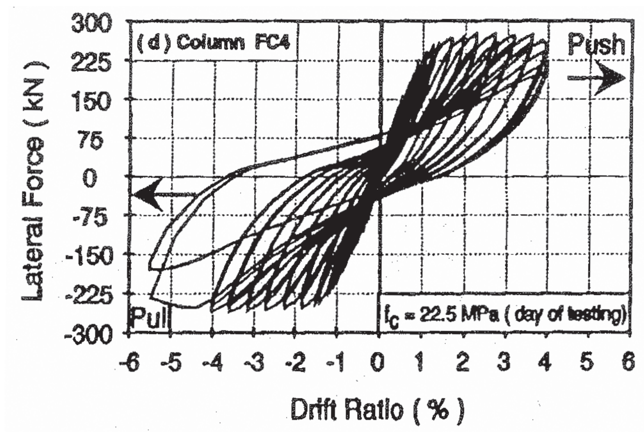


Fig. 2-10. Pinched hysteretic behavior typical of reinforced concrete elements.

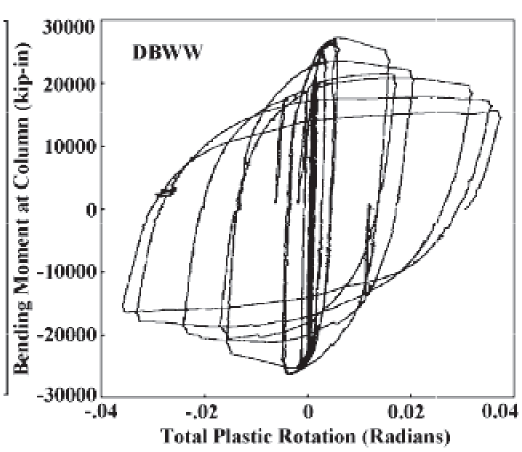


Fig. 2-9. Elastic-plastic strain degrading behavior of inadequately braced RBS moment connection.

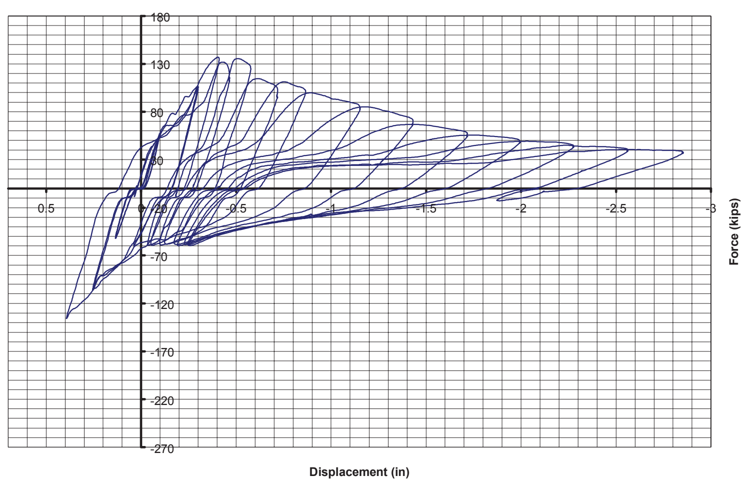


Fig. 2-11. Hysteretic behavior of brace loaded in compression beyond buckling limit state.

2.6 What is ductility?

Ductility is the property possessed by some structural elements, and structures composed of such elements, that enables them to sustain load-carrying capability when strained beyond their elastic limit. For structures that have well-defined yield and ultimate deformation capacities, such as those depicted in Figures 2-5 and 2-6, ductility, μ, is defined by Equation 2-12.

$$\mu = \frac{\delta_u}{\delta_y} \tag{2-12}$$

In this equation, δ_u and δ_y are the displacements at which failure and yielding, respectively, initiate.

Ductility is an important parameter for seismic resistance because it enables the design of structures that do not have adequate strength to resist strong earthquake shaking elastically to still survive such shaking through inelastic response. Structures that do not have ductility will fail when they are subjected to ground motion that deforms them beyond their elastic limit. Most of the design criteria contained in AISC 341 for design of the various types of steel and composite structures are intended to ensure that these structures will have sufficient ductility, enabling their design for forces that are substantially less than required to resist design ground motions elastically.

2.7 How does inelastic response affect a structure?

One of the principal benefits of inelastic response is that it limits the amount of force that is induced in the structure by the ground shaking. For example, if a structure has hysteretic characteristics similar to the elastic-plastic hysteretic behavior shown in Figure 2-5, no matter how far earthquake shaking deforms the structure it will never experience more force than F_y. If a structure is properly designed, this effect makes it possible to place ductile elements at key locations in the seismic load resisting system that will yield and protect other elements that are not ductile from being overstressed. This is a key strategy in design of structures for seismic resistance—sometimes called capacity design because elements in the structure that are not ductile are designed with sufficient capacity to resist the forces that will occur after the ductile elements yield.

Inelastic response also affects the amount of deformation a structure will experience in an earthquake. When a structure responds inelastically to earthquake shaking, a number of things can happen. If the structure is ductile, it will continue to provide resistance, after deforming beyond its yield point. However, its instantaneous stiffness will reduce, lengthening its effective periods of vibration and changing its mode shapes. In addition, as the structure strains inelastically, it will begin to dissipate a portion of the energy imparted to it by the earthquake in the form of strain energy.

The reduction in stiffness and period lengthening that accompanies ductile behavior tends to increase the amount of displacement the structure will experience as it is pushed by earthquake forces. At the same time, the inelastic strain energy that the structure dissipates acts as a form of damping and tends to reduce the amount of deformation induced by the shaking. Exactly how each of these behaviors will affect a specific structure depends on the initial dynamic characteristics of the structure and the dynamic characteristics of the ground motion. However, there are some general observations that can be made about the effect of inelastic response on the amount of deformation a structure will experience.

These effects tend to be different for structures having relatively long periods of vibration than for structures with short periods of vibration. For the purpose of this discussion, structures having a first mode period of vibration of 1 second or more can be considered long-period structures. Structures having first mode periods of 0.5 second or less may be considered short-period structures. Structures with fundamental periods between 0.5 and 1 second may behave either as short- or long-period structures, depending on the dynamic characteristics of the ground shaking.

In general, the displacement experienced by long-period structures that undergo inelastic response will be about the same as if the structure had remained elastic. This behavior was first noted by Newmark and Hall (1982) and is sometimes called the “equal displacement” rule.

Short-period structures behave in a different manner. When short-period structures yield, they tend to experience larger displacement than they would have if they remained elastic. If the hysteretic behavior of a short-period structure is such that it experiences pinching, this tends to increase the displacements still more.

Inelastic strength degradation tends to further increase inelastic displacement, both for short- and long-period structures. Strain hardening tends to reduce these displacements.

Regardless of whether a structure is brittle or ductile, or has short or long period, inelastic behavior will always result in structural damage. In steel structures, this damage will take the form of yielding, buckling and fracturing. Depending on the severity of this damage, it may or may not be necessary to repair the structure after the earthquake.

2.8 How does earthquake response cause collapse?

Earthquakes can cause structural collapse in several different ways. First, if the pieces of a structure are not adequately connected and “tied together,” the motions induced in the structure by earthquake shaking can allow these pieces to pull apart and, if one piece is supported by another, to collapse. This type of collapse is observed in bridges and other long structures that incorporate expansion joints. A portion of the Oakland–San Francisco Bay Bridge experienced this type of collapse in the 1989 Loma Prieta earthquake when the longitudinal displacement of a portion of the bridge deck exceeded the available bearing length (Figure 2-12).

Another way that earthquakes can cause structures to collapse is by overstressing gravity load bearing elements such that they lose load-carrying capacity. As an example, if the overturning loads on the columns in a braced-frame structure exceed the buckling capacity of the columns, these columns could buckle and lose their ability to continue to support the structure above. Such a failure occurred in the Piño Suarez towers in the 1985 Mexico City earthquake (Figure 2-13).

The third way that earthquakes cause collapse is by inducing sufficient lateral displacement into a building to allow P-Δ effects to induce lateral sidesway collapse of the frame. Sidesway collapse can occur in a single story, or can involve multiple stories. The collapse of the Kaiser Permanente Medical building during the 1994 Northridge earthquake is an example of a single-story collapse resulting from P-Δ effects (Figure 2-14). Often, it is difficult to distinguish these collapses from the local failures of elements previously described because the large displacements associated with sidesway collapse can often trigger concurrent local collapse.

2.9 How do structural properties affect inelastic response?

Stable inelastic response is the ability of a structure to resist ground shaking that stresses some elements beyond their elastic limit, without experiencing collapse. This occurs when a structure is capable of maintaining all—or at least most—of its post-yield lateral and vertical strength when deformed beyond its elastic limit. In addition to adequate initial strength and stiffness, important structural properties of members intended to undergo inelastic response include connections that are stronger than the members they connect—so that the members can develop their full strength—and member configurations that enable ductile post-elastic behavior.

In order to achieve ductile post-elastic behavior, it is necessary to avoid both global and local buckling of members. Local buckling is avoided by ensuring section compactness. Global buckling is avoided by providing lateral bracing.

Once a member yields, its effective modulus of elasticity—and, therefore, its resistance to buckling—decreases rapidly. In order to avoid premature onset of local buckling in members that undergo large inelastic deformations in the compressive range, it is necessary to have more restrictive limits on the width-thickness ratios of elements loaded in compression. The special compactness requirements contained in the AISC 341 for Special and Intermediate Moment Frames (SMF and IMF) are intended to achieve enhanced resistance to local buckling under inelastic cyclic behavior.

2.10 What are the most important aspects of seismic design?

A number of strategies are important to the design of structures that will behave adequately in strong earthquakes. These include provision of continuity, adequate stiffness and strength, regularity, redundancy, and a defined yield mechanism.

Continuity. All of the pieces that comprise a structure must be connected to each other with sufficient strength that when

Fig. 2-12. Partial collapse of the Oakland–San Francisco Bay Bridge, 1989 Loma Prieta earthquake. (Photo courtesy of P. Yanev)

Fig. 2-13. Collapse of the Piño Suarez Towers, 1985 Mexico City earthquake. (Photo courtesy of John Osterass)

the structure responds to shaking, the pieces don't pull apart and the structure is able to respond as an integral unit. An important aspect of continuity is having a complete seismic load resisting system so that a force that is applied anywhere in the structure has a means of being transmitted through the structure and to the foundation. In addition to vertical frames, a complete seismic load resisting system must also include horizontal diaphragms to transmit inertial forces to the vertical frames.

Stiffness and Strength. Structures must have sufficient stiffness so that the lateral deformations experienced during an earthquake do not result in P-Δ instability and collapse. Structures must have sufficient lateral and vertical strength such that the forces induced by relatively frequent, low-intensity earthquakes do not cause damage and such that rare, high-intensity earthquakes do not strain elements so far beyond their yield points that they lose strength.

Regularity. A structure is said to be regular if its configuration is such that its pattern of lateral deformation during response to shaking is relatively uniform throughout its height, without twisting or large concentrations of deformation in small areas of the structure. It is important to avoid excessive twisting of structures because it is difficult to predict the behavior of a structure that twists excessively. It also is important to avoid concentrations of deformations in structures because these concentrated deformations can become very large, leading to extreme local damage in the area of the concentration and a loss of vertical load-carrying capacity.

Redundancy. Redundancy is important because of the basic design strategy embodied in the building codes, which anticipates that some elements important to resisting lateral forces will be loaded beyond their elastic limits and will sustain damage. If a structure has only a few elements available to resist earthquake-induced forces, when these elements become damaged, the structure may lose its ability to resist further shaking. However, if a large number of seismic load resisting elements are present in a structure, and some become damaged, others may still be available to provide stability for the structure.

Defined Yield Mechanisms. Designing for a predetermined yield mechanism is perhaps the most important strategy. In this approach, which is often termed capacity design, the designer must decide which elements of the structure are going to yield under strong earthquake excitation. These elements are detailed so that they can sustain yielding without undesirable strength loss. At the same time, all of the other elements of the structure, such as gravity load-carrying beams, columns and their connections, are proportioned so that they are strong enough to withstand the maximum forces and deformations that can be delivered to them by an earthquake, once the intended yield mechanism has been engaged. In essence, the members that are designed to yield act as structural "fuses" and protect other elements of the structure from excessive force. This strategy ensures that critical members important to the vertical stability of the structure and its ability to carry gravity loads are not compromised.

Fig. 2-14. Single-story collapse of the Kaiser Permanente Office Building, 1994 Northridge earthquake. (Photo courtesy of P. Yanev)

SECTION 3
U.S. BUILDING CODE CRITERIA FOR EARTHQUAKE-RESISTANT DESIGN OF STEEL STRUCTURES

3.1 What codes and standards regulate design for earthquake resistance?

Regulation of building construction in the United States is generally the responsibility of local government, including individual cities and counties. Some states, such as California, adopt a statewide building code and require that all cities and counties within the state use this code as the basis for building regulation in their communities. Regardless, almost all cities, counties, states and territories that formally enforce building codes do so by adopting one of the model building codes, often with amendments intended to customize the model code to local conditions and practices. Most communities in the United States today base their building code on the IBC published by the International Code Council.

Although the IBC contains detailed provisions and requirements pertaining to fire/life safety, health, and other aspects of building design and construction, in recent editions, this code has adopted technical provisions related to structural design by reference to approved ANSI consensus standards. Under this code, the mandatory seismic design requirements for steel structures are therefore contained in a series of standards, including:

- SEI/ASCE 7, *Minimum Design Loads for Buildings and Other Structures*;
- AISC 360, *Specification for Structural Steel Buildings*;
- AISC 341, *Seismic Provisions for Structural Steel Buildings*;
- AWS D1.1, *Structural Welding Code, Steel*; and
- AWS D1.8, *Seismic Supplement to Structural Welding Code*.

In addition, the provisions of the nonmandatory standard, AISC 358, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications,* can be used to satisfy some of the design requirements for SMF and IMF contained in AISC 341. Although this compendium of codes and standards appears to be quite complex, generally, these documents are well coordinated with each other so that there are few overlaps or contradictions.

SEI/ASCE 7 is a loading standard that specifies the magnitude of all loads, including dead, live, wind, snow and seismic, that must be used, as a minimum for structural design, as well as the manner in which these loads must be combined for design purposes. Importantly for seismic design, SEI/ASCE 7 also is used to determine the Seismic Design Category for each structure, and thereby, the types of structural systems that can be used for seismic resistance. SEI/ASCE 7 identifies six different Seismic Design Categories, labeled A, B, C, D, E and F.

Seismic Design Category A encompasses those structures that, by nature of their geographic location and occupancy, pose negligible seismic risk, regardless of their design and construction. Seismic Design Category F encompasses those structures that present the most significant seismic risks, both because of their location, which will be within a few kilometers of a known active fault, and their occupancy, which is deemed as essential to the public health and safety in a post-earthquake environment. The other design categories cover structures of increasing seismic risk from B through E.

Steel structures assigned to Seismic Design Category A do not require specific design for seismic resistance. Rather, they need only have a complete lateral load resisting system capable of resisting 1% of the structure's dead weight, applied as a lateral force at each level. The design of steel structures is seldom controlled by this requirement, as the basic requirements for structural stability in AISC 360 generally represent a more severe design condition.

Steel structures assigned to Seismic Design Categories B and C must be designed with a complete seismic load resisting system, but this system does not have to incorporate any special seismic detailing requirements if the structure is designed using loads determined with a seismic response modification factor, R, having a value of 3. When an $R = 3$ system is not used, the seismic load resisting system must be designed to comply with specific seismic detailing criteria. The detailing criteria are organized by structural system type. For steel and composite steel and concrete structures, the available system types are:

- Concentrically braced frames;
- Eccentrically braced frames;
- Buckling-restrained braced frames;
- Moment frames;

- Shear walls; and
- Dual systems containing both moment frames and shear walls or braced frames.

Within these broad categories, there are also a series of subcategories, termed Ordinary, Intermediate and Special, that relate to the amount of ductility provided when properly designed and detailed. The detailing and other special design requirements for these various seismic load resisting systems are contained in AISC 341. Engineers designing steel structures in Seismic Design Category A, or using an R = 3 system in Seismic Design Categories B and C need not refer to AISC 341.

Regardless of the Seismic Design Category that is assigned to a structure, the available strength, either in Allowable Stress Design (ASD) or Load and Resistance Factor Design (LRFD) format, is determined in accordance with AISC 360, as are the basic construction requirements. AWS D1.1 controls the requirements for design, execution and quality assurance of structural welding in steel structures, as well as some aspects of thermal cutting of structural steel. AWS D1.8 provides supplemental welding criteria applicable only to the seismic load resisting systems of structures designed in accordance with AISC 341. The Research Council on Structural Connections (RCSC) *Specification for Structural Joints Using ASTM A325 or A490 Bolts* provides the criteria for design and workmanship associated with high-strength bolting.

AISC 358 can be used to satisfy requirements pertaining to the design, detailing and fabrication of special and intermediate moment frames, however, its use is not mandatory.

The American National Standards Institute (ANSI) is the oversight body that sets the development and maintenance requirements for ANSI consensus standards. Among other requirements, the developers of ANSI standards, such as SEI/ASCE and AISC, must review each standard and either adopt an updated edition of the standard or reaffirm the validity of the prior standard, at least every five years. In the past, these organizations typically revised their standards more often than this, but there is a growing trend towards conformance with the five-year development schedule.

The International Code Council (ICC) updates and republishes the IBC every three years. In addition, it publishes supplements every 18 months. Generally, from a perspective of structural requirements, the updates and supplements to the IBC are limited to adoption of the latest editions of the referenced ANSI standards. On occasion the code will adopt supplemental provisions to those contained in the standards. This typically occurs when one or more structural failures or other incidents point out a significant deficiency in the standards that the standards development organizations do not have time to address in time for the next building code edition.

Since communities do not always adopt the latest edition of the model building codes, and because there are several model building codes in use, engineers should ascertain which code is actually in effect in a community prior to undertaking a design and use that code, as well as the editions of standards specifically referenced by that code. While most building officials are open to acceptance of designs that are conducted in accordance with updated codes and standards, this is not always the case. It is also important to recognize that updated editions of industry standards may conflict with requirements of older codes, so it is important to exercise caution when newer versions of code-referenced standards are used with an outdated code.

3.2 What are the earthquake performance objectives of U.S. building codes?

In general, governments adopt and enforce building codes through the police powers that enable them to protect the public health, safety and welfare. The first U.S. building codes were adopted by major cities in the late 19th century and were intended to reduce the risk of urban conflagration and the attendant large life and property losses that occurred with events like the great Chicago fire of 1871. Gradually, these codes were expanded to address sanitation, ventilation and structural stability under various load conditions.

Earthquake requirements began to appear in building codes in the western United States in the 1920s. At that time, scientific knowledge of the causes of earthquakes, their effects and the response of buildings to earthquakes was quite limited. The earthquake requirements contained in the code were generally empirical. When earthquakes occurred, engineers would observe the types of construction that performed well and the types that performed poorly, and then incorporate rules in the building code that would prohibit poorly performing construction and encourage types of construction observed to perform in a satisfactory manner. Since, even in California, damaging earthquakes are a relatively infrequent event, affecting a city perhaps one time every 20 years or more, rather than trying to avoid damage completely, these early building codes were intended to protect life safety by encouraging the construction of collapse-resistant structures.

The protection of life safety, through the avoidance of earthquake-induced collapse, remains the primary goal of U.S. building codes today. However, since the mid-1970s, the building codes have adopted supplemental objectives based on building occupancy. The IBC and SEI/ASCE 7 assign each structure to one of four Occupancy Categories used to differentiate between the expected performance of structures and the relevant design requirements.

- Occupancy Category I encompasses structures, the failure of which, would pose little risk to the public as they are seldom occupied by persons and are not located where their failure is likely to injure people. This includes most barns and certain other agricultural and industrial structures.

- Occupancy Category II encompasses buildings of average risk to public safety including most residential, commercial, institutional and industrial structures.

- Occupancy Category III encompasses structures that pose a higher than ordinary risk to the public safety—either due to the very large numbers of persons housed within the structure or the limited mobility of persons within the structure. Many very tall structures, sports venues, convention centers and some schools fall into this category.

- Occupancy Category IV encompasses structures that would pose an intolerable risk to the public in the event of failure, such as the loss of emergency post-earthquake response capability or the release of large quantities of toxic materials. Generally, hospitals, police and fire stations, emergency communications centers and certain industrial facilities fall into this category.

For Occupancy Category II structures, the basic goal of both the IBC and SEI/ASCE 7 is that structures assigned to this category have limited risk of collapse, on the order of 10% or less, in the event that they experience Maximum Considered Earthquake (MCE) effects. For most sites in the United States, MCE effects have an average return period of 2,500 years. For sites located within a few kilometers of known active faults, however, MCE effects are set by the codes and standards based on a conservative estimate of the shaking likely to occur from a maximum magnitude event on these faults.

Regardless of the location of a building, the code has no expectation of damage control, beyond collapse avoidance for the MCE event. If a building experiences such shaking, it is likely to experience severe damage to the structural frame and possibly extreme damage to nonstructural components and systems. There is some risk of injury and limited risk of life loss as a result of such damage.

In addition to the basic collapse avoidance goal set for Occupancy Category II structures, the code also has an objective that nonstructural components essential to life safety in buildings of this occupancy category, including stairs, fire sprinkler systems, emergency egress lighting and piping systems housing toxic materials, remain functional for Design Earthquake shaking, defined as shaking having an intensity that is two-thirds that of MCE shaking. Occupancy category II structures are expected to experience both structural and nonstructural damage if they experience Design Earthquake shaking; however, the risk to life safety should be minimal. It is expected but not guaranteed that such damage would be repairable, but there is no expectation that these buildings would be habitable until repaired. The probability of actually experiencing Design Earthquake shaking varies around the United States from perhaps one time in a thousand years in areas of low earthquake risk to a few hundred years in regions of highest risk.

It is expected that structures that are properly designed, constructed and maintained to the code requirements for Occupancy Category II would be able to resist relatively frequent, low-intensity earthquakes without significant damage or loss of occupancy. The exact frequency of occurrence of events that cause such limited damage is not well defined. For regions where damaging earthquakes occur frequently, such as those in portions of Alaska, California and Washington, this may be on the order of 25 to perhaps 50 years. For sites in regions of lower earthquake risk, this may be on the order of several hundred years.

The performance objectives for structures of other Occupancy Categories are related to those for Occupancy Category II structures. Those for Occupancy Category III structures are essentially identical to those for Occupancy Category II structures, except that they are expected to achieve these objectives with higher reliability. Thus for MCE shaking, there is expected to be significantly less likelihood of collapse, perhaps on the order of 5% or less. The risk of life safety endangerment or occupancy loss would be similarly reduced relative to that for Occupancy Category II structures. For Occupancy Category IV structures, the risk of collapse, given MCE shaking is very small, on the order of 3% or less. It is expected that these structures would retain their ability to function for most earthquakes likely to affect them, which in regions of highest earthquake risk, might include events having return periods of 100 years or so. For regions of lower earthquake risk, this might extend to earthquake events having return periods of 500 to 1,000 years. Occupancy Category IV structures will experience damage in earthquakes, but to a lesser extent than structures designed in accordance with the requirements for Occupancy Categories I, II or III.

Occupancy Category I structures have a significant risk of collapse if they experience MCE shaking, perhaps on the order of 20%. The risk of damage and occupancy interruption is substantial for these structures, even for relatively frequent, low-intensity earthquakes.

The factors that affect how a building will actually perform in an earthquake include:

- The specific characteristics of the earthquake itself (that is, the place along a fault at which the earthquake originates and the direction and speed at which the fault rupture occurs);

- The nature of the rock and soils through which the shaking passes on its way to the building site;

- The duration of strong shaking;
- The adequacy of the design and construction of the individual building; and
- The building's condition at the time the earthquake occurs.

These factors are impossible to precisely predict. As a result there can be substantial variation between the expected and actual performance of a structure when an earthquake occurs. Most structures will perform better than anticipated by the code's performance objectives, while others will perform worse.

3.3 Why have recent building codes expanded the areas of the country requiring seismic design?

For many years, earthquakes were a serious design consideration only in the western United States, and in a few eastern cities like Boston, that considered that they had significant earthquake risk. Engineers in most of the United States did not have to design their structures for earthquake resistance and had little knowledge of earthquake engineering. This changed in 2000 with the publication of the IBC (and its referenced standards), which requires seismic-resistant design for much of the United States and adopts a different model of acceptable risk than was contained in earlier codes.

Prior to the publication of the 2000 IBC, U.S. building codes based requirements for seismic design on the 500-year earthquake risk, as mapped by the USGS in the 1970s. According to those seismic risk maps, most sites in coastal California, Oregon and Washington, and the intermountain region extending from Salt Lake City to Idaho, had significant risk of experiencing Modified Mercalli VII or more intense ground shaking at least one time every 500 years. The building codes required buildings on sites with this risk to be rigorously designed for earthquake resistance. Buildings on sites with lower risk were either not required to be designed for earthquake resistance or were required to be designed to relaxed standards, which often could be satisfied by basic wind load design requirements. While the risk of major earthquakes in the New Madrid fault zone and near Charleston, South Carolina, were recognized, the USGS felt that earthquakes in these regions only occurred very infrequently on the order of perhaps one time every 1,000 to 2,000 years.

When ICC was formed in the mid-1990s and announced their intent to publish a new building code, the Building Seismic Safety Council (BSSC), a nonprofit council of the National Institute of Building Sciences (NIBS), and the USGS were funded by the Federal Emergency Management Agency (FEMA) to develop a new series of seismic risk maps for use in the new code. BSSC and USGS convened a group of structural engineers and seismologists from around the United States to provide guidance to the USGS in developing the new maps. This group was called the Project 97 working group, since they were convened in 1997.

The Project 97 group determined that if the code continued to base earthquake design requirements on 500-year shaking, a repeat of the 1811–1812 New Madrid or 1886 Charleston earthquakes would cause widespread collapse of structures in the affected regions, and thousands of fatalities. In order to avoid such disasters, the group decided that, rather than designing for life safety protection in the event of 500-year shaking, as had been the basis for earlier codes, the new code should revise its objectives to provide protection against collapse in the most severe events ever likely to affect a region. Thus, the 500-year maps were abandoned in favor of MCE shaking maps that were based either on 2,500-year shaking in most of the United States or estimates of the maximum probable shaking near major active faults in others.

Another major change that the Project 97 working group recommended was to include consideration of local soil amplification effects on ground shaking when determining the level of seismic design that is required. Whereas the older codes assigned regions of the country to a seismic risk category based on the intensity of motion expected on firm soil sites, the new maps and new code made this assignment considering the effects of local soil conditions on shaking intensity. The combined result of these two decisions (lengthening of the return period for the maps from 500 to 2,500 years and including site soil effects in the calculation of shaking intensity) resulted in the great expansion of seismic design requirements throughout the United States. In essence, the Project 97 working group decided that the older design approach left too many people at risk of a major earthquake disaster, in repeats of events that had occurred in historic times, and attempted to reduce this risk.

3.4 What is Site Class, and why is it important?

When an earthquake occurs, the primary factors that affect the intensity of shaking that is experienced at a building site, and the destructive potential of this shaking, are:

- The magnitude of the earthquake;
- The distance of the site from the fault;
- The direction of fault rupture;
- The characteristics of the rock through which the earthquake shaking propagates as it approaches the site; and
- The nature of soils at the site.

In general, soft compressible soils tend to amplify shaking with long-period (0.5 second or higher) content, and to attenuate motion with short-period content. Conversely, firm, relatively incompressible soils tend to attenuate long-period motion.

The building codes and SEI/ASCE 7 adopted the concept of Site Class as a means of categorizing the tendency of a site to amplify or attenuate motion in different period ranges, in a relatively simple manner. Since the characteristics of soil within the upper 100 meters (30 ft) relative to the ground surface have the most significant effect on the shaking that is significant to buildings and building-like structures, Site Class is determined based on the average properties of soil within this zone. Six different site classes are designated in the code and are labeled A, B, C, D, E and F.

- Site Class A corresponds to very hard and competent rock including granites, quartz and similar stones.
- Site Class B corresponds to soft sedimentary rocks including sandstone, claystone, siltstone and similar materials.
- Site Class C corresponds to firm site conditions typified by dense sand and gravels and very stiff clays.
- Site Class D corresponds to average site conditions containing moderately dense granular soils and stiff clays.
- Site Class E corresponds to soils having high plasticity and compressibility, notably including weak clays, loose saturated silts and similar materials.
- Site Class F corresponds to soils that are unstable and which could experience such effects as liquefaction.

SEI/ASCE 7 prescribes procedures to use measurable physical characteristics of the site soils, including shear wave velocity and shear strength, to determine to which class a site should be assigned. If site-specific study is not performed, the code permits the assumption that a site conforms to Site Class D, as long as it does not conform to Class E or Class F.

3.5 What are the advantages of a site-specific seismic hazards study?

There are two basic components to a site-specific seismic hazard study.

- A probabilistic determination is made of the intensity of shaking at the surface of bedrock at a site, as a function of return period. This is conducted by considering each of the faults known to exist in a region and estimating the magnitudes of earthquakes that are likely to occur on these faults, the probability of experiencing such an earthquake on each fault, and the probable intensity of ground shaking in bedrock at the site, given that these earthquakes occur. This is the process that the USGS followed in developing the national seismic hazard maps contained in the building code. When the USGS made these maps, however, they performed this exercise for every site on a 1-kilometer by 1-kilometer grid, across the United States, rendering them somewhat inaccurate for some sites. Also, as seismologic knowledge increases, seismologists may alter their opinion of the likely earthquakes on some faults, and the frequency with which they occur. Thus, site-specific hazard analyses can provide an improved assessment of the likely shaking at a site.
- A site response analysis is made and used to determine the intensity of ground shaking at the ground surface, as shaking propagates upward from the top of bedrock to the ground surface. This can be used to more accurately reflect the effects of Site Class on shaking intensity than can be accomplished using the default Site Class categories contained in SEI/ASCE 7. Site response analysis can also be used to determine if ground improvement techniques, such as soil densification, can effectively reduce the intensity of shaking effects structures will experience.

For most sites and projects, it is not necessary to perform site-specific seismic hazard studies because the ground motion parameters obtained using the maps contained in the code and the Site Class coefficients provide sufficiently accurate results. However, sometimes the use of site-specific hazard analyses will allow the design forces for a structure to be reduced by as much as 20% through more accurate representation of the hazard associated with nearby faults, and of site effects.

For large projects, this can result in substantial economic savings. It is also common to perform site-specific hazards studies for very important structures, such as power plants, petroleum refineries, and very tall buildings, in order to ensure that the designs are adequate to ensure good performance of the structures in the event an earthquake actually occurs.

3.6 What are Seismic Design Categories, and how are they determined?

Classification by Seismic Design Category is a procedure used by the building code to regulate the amount of risk to society that is posed by earthquake-induced failure of the structure. This risk is a function of several factors, including:

- The probability that the building site will experience intense ground shaking;
- The resistance of the structure to this ground shaking; and,
- The consequences of failure, should it occur.

The code uses two of these factors—the risk of experiencing intense ground shaking and the consequences of failure—to determine the Seismic Design Category for a structure. Once the Seismic Design Category is determined, the code regulates the resistance of the structure to earthquake-induced failure through various design and detailing measures.

The risk of experiencing intense ground shaking is determined based on the values of response acceleration coefficients obtained from the national seismic hazard maps, adjusted for Site Class. The consequences of structural failure are determined based on the Occupancy Category (see Section 3.2).

Six Seismic Design Categories, labeled A, B, C, D, E and F, are recognized by the code.

- Seismic Design Category A includes all structures that are not expected to experience destructive levels of ground shaking, regardless of their occupancy.
- Seismic Design Category B structures are Occupancy Category I, II or III structures that may experience shaking of Modified Mercalli Intensity VI. These structures must be designed for the forces that such shaking would generate, but since the shaking is not very intense, the code permits the use of systems with limited ductility.
- Seismic Design Category C generally corresponds to Occupancy Category I, II or III structures that may experience shaking of intensity VII. In addition to requiring design of the structures for the resulting forces, the building code also requires that nonstructural components essential to life safety in such buildings be anchored and braced for seismic resistance.
- Seismic Design Category D generally corresponds to structures of Occupancy Category I, II or III that may experience intensity VIII or IX shaking. The code requires that these structures be provided with ductile seismic load resisting systems, more extensive protection of nonstructural elements and rigorous construction quality assurance measures.
- Seismic Design Category E corresponds to Occupancy Category I, II or III structures that are located within a few kilometers of major active faults, which can experience intensity X or higher shaking. In addition to all the requirements for Seismic Design Category D, such structures must be designed to conform to restrictive limits on irregularity.

Structures in Occupancy Category IV would pose a significant and unacceptable risk to society if they were to experience earthquake failure. Therefore these structures are assigned to the next higher Seismic Design Category than would be required if they were a lower Occupancy Category. Thus, an Occupancy Category IV structure located on a site expected to experience a maximum of intensity VII motion would be assigned to Seismic Design Category D, rather than C. Seismic Design Category F corresponds to Occupancy Category IV structures located close to major active faults.

3.7 What are Special, Intermediate and Ordinary seismic load resisting systems?

It would not be economically practical to design most structures to be able to resist very severe earthquakes without damage, since such earthquakes may never occur during the building's useful life. Therefore, the design procedures contained in the code intend that structures be designed to sustain limited levels of damage, while still attempting to avoid collapse. Structures that have regular distributions of mass, stiffness and strength, and seismic load resisting systems that are detailed to sustain cyclic straining beyond their elastic limit without loss of load-carrying capacity are able to perform acceptably in this manner. Structures that do not have these characteristics will experience severe damage and might collapse, unless they are designed with adequate strength.

Over the years, engineers and researchers have observed that structures that are proportioned and detailed to have regular configuration with uniform distribution of deformation; well-defined zones of yielding and inelastic behavior and adequate detailing to permit this inelastic behavior to occur without loss of load-carrying capability can undergo extensive inelastic deformation without collapse, even when designed with relatively low strength. As a structure's detailing and configuration characteristics deviate from these ideals, its ability to withstand strong shaking without collapse requires greater and greater strength.

In recognition of this, the design procedures contained in the building code regulate the amount of lateral strength a structure must have based on the configuration and detailing characteristics of its structural system. Structures that have desirable characteristics for resisting strong ground shaking inelastically are termed as Special; structures that have low capacity to resist strong shaking in an inelastic manner are termed Ordinary. Structures with characteristics between these two classifications are termed Intermediate.

AISC 341 prescribes specific detailing requirements for braced-frame structures, moment-frame structures, shear-wall structures, and dual-system structures, which must be followed to be categorized as Ordinary, Special, and Intermediate. SEI/ASCE 7, which regulates the required strength of structures for earthquake resistance, sets this strength based on whether the system is Special, Intermediate or Ordinary. Special structures can be designed for greatly reduced forces relative to Intermediate and Ordinary structures, because of their presumed capacity to withstand greater inelastic demands. SEI/ASCE 7 also places limits on the Seismic Design Categories and heights of structures for which these different systems can be used.

3.8 How are design seismic forces and drifts determined?

SEI/ASCE 7 permits determination of seismic design forces and drifts by any of several different methods. Generally these can be classified as elastic static methods, elastic dynamic methods and a nonlinear dynamic method. The nonlinear dynamic method is conducted by performing a numerical integration of the equation of motion (see Section 2.2), with the stiffness matrix for the structure modified at each time step to account for damage that has occurred. This procedure is seldom used for the design of new structures, because it is complex and time consuming to perform; it results in the creation of a great deal of data, which must be processed and interpreted; the code does not specify acceptance criteria for this method, which can be used to demonstrate that a design is acceptable; and the code requires third-party peer review when this procedure is followed. However, the nonlinear dynamic method is used more often for the evaluation and upgrade of existing structures because it is less conservative than the linear approaches that are commonly used for design of new construction and can provide significant economic advantage for existing buildings.

All of the linear methods that are commonly used to obtain design seismic forces and displacements are based on the use of linear acceleration response spectra (see Section 2.4). SEI/ASCE 7 contains a procedure to derive a design linear acceleration response spectrum for any site in the United States, using the response acceleration parameters determined from the national seismic hazard maps contained in the code and coefficients that adjust the response accelerations for Site Class effects.

Figure 3-1 is a typical design response spectrum derived using the code procedures. The horizontal axis of this plot is the structure's natural period, T. The vertical axis of the spectrum is the response acceleration, S_a. Three parameters are used to fully define this response spectrum curve: S_{DS}, the design spectral response acceleration for short periods; S_{D1}, the design spectral response acceleration at 1-second period; and, T_L the period at which the spectrum transitions from constant response velocity to constant response displacement. The value of T_L is obtained directly from a map in the code, and the values of S_{DS} and S_{D1} are derived using mapped acceleration values and parameters related to the Site Class (see Section 3.4).

The design response spectrum, as indicated in Figure 3-1 does not actually represent any one specific earthquake. Rather, it represents the envelope of the series of spectra that could affect the building site, at the probability of the design earthquake. It is also possible to use site-specific spectra, rather than this general design spectrum contained in Figure 3-1. Site-specific spectra generally have a more rounded appearance and a narrower range of period over which the very high response accelerations represented by S_{DS} in Figure 3-1 will occur.

The basic method of determining lateral design seismic forces and displacements is the so-called equivalent lateral force procedure. This consists of a simple, first-mode, elastic response spectrum analysis. The first step in this process is to determine the structure's first-mode natural period of vibration, T. The code provides a series of equations, applicable to different structural systems that permit rapid estimation of the period based on a building's height above grade. These approximate period equations usually provide a conservatively low estimate of the period.

As an alternative to the use of these approximate period equations, the code permits direct calculation of the period using methods of structural mechanics. Many structural analysis programs commonly used in design offices can perform this calculation. However, the code places an upper limit on the value of calculated period to reduce the risk that an inappropriate modeling of a structure will lead to calculation of an excessively long period.

Once the period is determined, the design spectral response acceleration at the fundamental period of the structure, $S_a(T)$, is determined from the design response spectrum (Figure 3-1). The total lateral seismic design force, V, for the structure, is then determined from Equation 3-1.

$$V = \frac{S_a(T)}{R/I} W \tag{3-1}$$

In this equation, W is the seismic weight of the structure, I is the Occupancy Importance Factor and R is the seismic response modification factor.

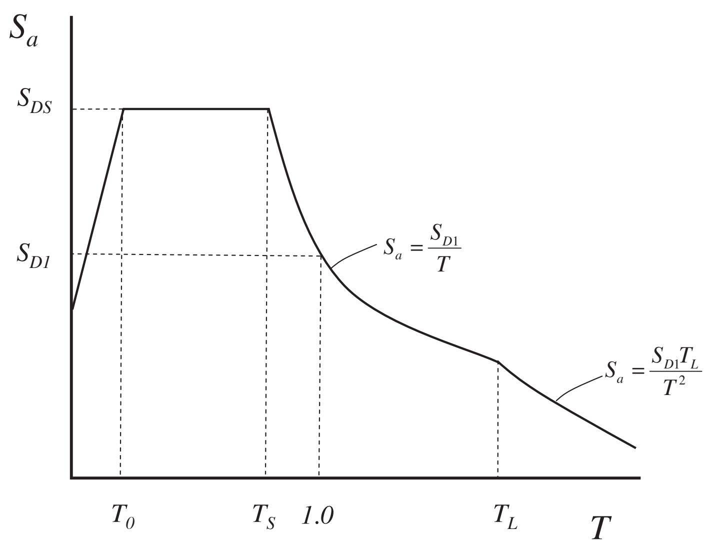


Fig. 3-1. Design response spectrum.

The seismic weight typically includes the building's dead load, the weight of any permanent equipment, and an allowance for partitions if the building is in an occupancy that uses demountable partitions. Live load, other than partitions, is not typically included in the seismic weight, because usually, only a small portion of a building's design live load will be present when a large earthquake occurs and also because the live load represents the weight of things that are not typically firmly attached to a structure, and which, therefore, may not be fully effective in the structure's response. The Occupancy Importance Factor is determined from a table in SEI/ASCE 7, based on the building's Occupancy Category.

The seismic response modification factor, R, is determined from a table in SEI/ASCE 7, based on the type of structural system that is used. For systems that are detailed for ductile behavior and significant inelastic response capacity, usually designated as Special systems, the value of R can be relatively large, on the order of 8. For systems that have lower quality detailing, a lower value of R is used. In this manner, the code specifies higher design forces as system ductility decreases.

The total seismic design force, V, is distributed to the various stories of the structure in accordance with Equation 3-2.

$$F_x = \frac{w_x h_x^{\,k}}{\sum w_i h_i^{\,k}} \tag{3-2}$$

In this equation, F_x is the lateral seismic force applied at level x; w_i and w_x are the seismic weights of the structure at levels i and x, respectively; h_i and h_x are the heights of the structure at levels i and x, respectively; and k is a coefficient related to the building's period, T, that varies between a value of 1 to 2 to account for the effects of higher modes in longer period structures.

The design lateral forces, F_x, are applied to an analytical model of the structure, and the forces in each of the members resulting from this analysis constitute the seismic design forces for the member resulting from horizontal ground shaking. These design forces do not actually represent the forces that will occur in the structure when responding to design earthquake shaking. Rather, they are a tool used by the code to set the minimum permissible strength of a structure. Ground shaking that is substantially less intense than the design level shaking (reduced by a factor R/I) will produce this level of forces in the structure. If the structure is efficiently designed, so that its actual strength closely matches the minimum design strength specified by SEI/ASCE 7, it will begin to yield when it experiences this level of ground shaking.

Most structures have sufficient redundancy that a number of elements must yield before a full yield mechanism will occur. Typically for such structures, the amount of lateral force necessary to cause a full yield mechanism will be significantly—on the order of two to three times—larger than the design forces. The ratio of the lateral forces that cause the formation of a full yield mechanism in the structure to the lateral forces that cause first yielding is termed the overstrength and is represented in the code by the symbol Ω_o. The code requires that some members, the failure of which could lead to structural collapse must be proportioned to resist the full overstrength of the structure. The code permits this overstrength to be approximated by the expression $\Omega_o E$, where the value of coefficient Ω_o is specified in SEI/ASCE 7 based on the selected structural system, and E represents the design lateral seismic forces in the member under the design seismic lateral forces, F_x.

In addition to the lateral seismic design forces, the code also requires that structures be designed for a vertical seismic design force to account for the effect of vertical ground shaking. The vertical seismic design force is computed as a fraction of the structure's dead weight, calculated as $0.2S_{DS}D$.

The design drift for a structure is determined by applying the lateral forces, F_x, computed using Equation 3-2, to a model of the structure, performing a linear static analysis and determining the lateral displacement at each floor i, Δ_i. The design story drift in story x, δ_x, is determined using Equation 3-3.

$$\delta_x = \frac{C_d \delta_{xe}}{I} \tag{3-3}$$

In this equation, δ_{xe} is the difference in lateral displacements of the floors immediately above and below the story, I is the Occupancy Importance Factor previously discussed, and C_d is a deflection coefficient specified by SEI/ASCE 7 based on the selected structural system. The purpose of the deflection coefficient is to adjust the deflections computed under the design seismic forces to a level that corresponds to an estimate of the real inelastic displacement anticipated during design earthquake shaking. The value of the deflection coefficient, C_d, is typically similar to, but slightly less than, the value of the seismic response modification factor, R, used to calculate the design seismic forces. It is adjusted by the Occupancy Importance Factor, I, in Equation 3-3 to account for a similar adjustment in the R values inherent in the base shear equations.

The code permits the use of the basic procedure described earlier for all structures in Seismic Design Categories B and C, and most structures in Seismic Design Categories D, E and F. However, the code does require that the seismic design forces for structures in Seismic Design Categories D, E and F that either have very long fundamental periods or certain types of configuration irregularities be determined using either the modal response spectrum procedure or the response history analysis procedure. The modal response spectrum procedure is similar to the equivalent lateral force procedure described earlier, except that the response of the structure in each significant dynamic mode is considered.

The mathematical computations associated with this procedure, though not complex, are tedious and so are usually performed using a computer. Most structural analysis software used in design offices today has the capability to perform these response spectrum analyses.

3.9 Why does the code impose drift limits on buildings?

There are two basic reasons that the code places limits on the design drift for a building. The first of these relates to a lack of confidence in our ability to reliably predict structural response and behavior under very large deformations. In effect, the drift limits force the design of structures that should respond within our zone of comfort. A second reason the code limits drift is to protect nonstructural components, including exterior curtain walls, interior partitions and similar items, against damage during moderate intensity ground shaking. This latter purpose is the reason SEI/ASCE 7 specifies more restrictive drift limits for structures in Occupancy Categories III and IV than it does for structures in Occupancy Categories I and II.

3.10 What is redundancy?

Redundancy is a property of a structure that exists when multiple elements must yield or fail before a collapse mechanism forms. This is an important property of structures that are designed to provide earthquake resistance because the basic design philosophy assumes that design-level shaking will cause significant damage to the structure. If a structure is designed in such a way that the failure of a single element—or even a few elements—will lead to collapse, it will not be able to sustain damage and still perform acceptably as anticipated by the code.

Following the 1994 Northridge earthquake, engineers investigating damaged buildings discovered that many recently designed structures had little inherent redundancy. This was a result of transitions in market condition with increases in the cost of labor, and also the availability of members with large section properties that could provide all of a structure's required strength with a few members. Engineers observed a direct link between the level of redundancy inherent in structures and the severity of damage sustained. Following this discovery, the building code was modified to require consideration of redundancy as part of the design process for structures assigned to Seismic Design Categories D, E and F. In such structures, a redundancy coefficient, ρ, is determined based on the number and placement of seismic load resisting elements in each story. For structures with low inherent redundancy, the required seismic design forces are amplified to ensure that the structure is stronger and more resistant to damage.

3.11 What are the advantages of distributed structural systems?

Distributed systems—structures configured with lateral resistance spread throughout the structure—provide several important benefits. The first of these is that distributed systems are inherently more redundant (see Section 3.10). Another important benefit of distributed systems is that because the size of individual members in the framing tends to be smaller, connections are also smaller and more economical. Finally, in some structural systems, smaller members tend to have greater ductility than larger members, due to effects of scale.

3.12 What is an irregularity, and why is it important?

The structures that perform best in response to earthquakes, in addition to having adequate strength, stiffness, ductility and redundancy, also have distributions of stiffness, mass and strength that enable them to respond to earthquake shaking with deformations well distributed throughout the structure, rather than concentrated in one or a few locations. The reason these characteristics are important is that if inelastic deformation is concentrated in a few locations within a structure, the inelastic capacity of the elements in these areas of large deformation demand can easily be exceeded, and instability can develop.

SEI/ASCE 7 defines several types of irregularities, which are generally classified as horizontal irregularities or vertical irregularities, depending on the nature of the irregularity. Most of these irregularities relate to the geometric configuration of the structure and in particular, its seismic load resisting system.

The code prohibits the design of structures with some types of irregularities in Seismic Design Categories D, E and F, including:

- Extreme Soft Story Irregularities, which occur when the lateral stiffness of a story is substantially less than that of the stories above;
- Extreme Weak Story Irregularities, which occur when the lateral strength of one story is substantially less than that of the story above; and,
- Extreme Torsional Irregularities, which occur when the arrangement of vertical elements of the seismic load resisting system permit the structure to twist extensively as it translates.

The code permits structures with certain other types of irregularities in these Seismic Design Categories, but requires the use of dynamic analysis to determine the design seismic forces and deformations for such structures. This is because the simplifications inherent in the equivalent lateral force technique of analysis cannot adequately capture the concentrations of forces and deformations that occur in structures with these irregularities.

3.13 What is expected strength?

Different steel members conforming to the same specification and grade of steel can have significantly different yield and tensile strengths. This is because each heat of steel that is produced has a slightly different chemistry and also because the yield strength of a steel member also depends on the member's geometry, the thickness of its webs and flanges, the member's thermal history, and the amount of cold working that occurs as the member is formed and then fabricated. The minimum-specified values that are associated with standard specifications such as ASTM A992 and A572 are, as the name implies, minimum acceptable values. Most structural steel conforming to one of these specifications will have strengths that exceed these minimums. Although the possible variation of strength varies from one grade of steel to another, depending in part on the limitations contained in the standard specification, coefficients of variation on the order of 20% of the minimum specified value are not uncommon. This means that more than half of the structural steel used on projects is likely to have a strength that is 20% higher than the minimum value and some such steel will have a strength that is nearly 40% higher than the minimum.

While additional strength is not of concern in design for most types of load conditions, it can be a problem in seismic design. This is because in seismic design, particularly for structural systems designated as Special or Intermediate, the design is based on an assumption that yielding and other inelastic behavior will occur in specific portions of the structure. For example, in braced-frame structures, it is desirable that inelastic behavior is accommodated in the form of yielding of tensile braces rather than failure of bolted connections. AISC 341 requires that connections in such structures must be designed so that the connection is able to develop the yield strength of the braces. In order to do this, the designer must have an understanding of how strong the member is likely to be.

AISC 341 uses the concept of expected strength to provide designers with an estimate of the probable effects of steel strength variation on the strength of individual members. The specification defines a coefficient, R_y, that is a measure of how much stronger, on average, typical steel sections are than the minimum-specified value. The quantity R_yF_y is termed the expected strength and is used to estimate the maximum force that a member is likely to be able to develop before it yields, so that other members and connections that are not supposed to yield can be designed with a strength that exceeds this force. In some provisions, the expected strength includes an additional factor that is intended to account for the effects of strain hardening of the steel, which further elevates the effective yield strength.

3.14 What is capacity design?

Capacity design is an approach used to design structures for seismic resistance in which the strength of the members comprising the seismic load resisting system are proportioned such that inelastic behavior is accommodated in specific designated locations that are adequately detailed to accommodate this behavior. When these elements yield, they limit the force that can be transmitted to other elements, effectively shielding them from overstress and allowing them to resist design earthquake excitation while remaining elastic. This practice permits the elements that are not expected to yield or experience inelastic behavior to be designed and specified without rigorous detailing practices intended to provide ductile behavior.

Many of the provisions contained in AISC 341 are intended to produce a capacity design of the seismic load resisting systems. As an example, Special Moment Frames must be proportioned such that inelastic behavior is accommodated through plastic hinging within the spans of the beams in moment frames, rather than the columns. As another example, Special Concentrically Braced Frames must be designed such that the connections and columns are stronger than the braces so that inelastic behavior is accommodated through yielding and buckling of the braces.

3.15 What is overstrength?

Overstrength is a measure of the additional strength, over and above the minimum, required to resist code-specified forces that almost all structures posses to some degree. There are a number of sources of overstrength in structures. First, there is material overstrength related to the inherent variability in our materials of construction. The minimum-specified strengths associated with a particular grade of steel, as with all materials, are just that—minimum permissible values. Most materials that are actually incorporated in a structure will be somewhat stronger than the minimum value permitted by the specifications. In the case of structural steel, material overstrengths on the order of 10% are common, and some grades of steel, such as ASTM A36, can have overstrengths as much as 30% of the minimum specified value. See Section 3.13 for additional discussion of this source of overstrength.

Another important source of overstrength is the tendency of structures to exhibit sequential yielding. Consider even a very simple structure, such as a one-bay, one-story moment frame with pinned column bases. Figure 3-2 shows hypothetical moment diagrams for this structure under (a) gravity loading, (b) lateral loading, and (c) combined gravity and lateral loading. When this structure is designed, the beams and columns must be designed with adequate strength to resist the combined gravity and lateral moments. However, under actual lateral loading, this moment demand will be experienced at one corner of the frame before the other, because the combined moment diagram is significantly larger at this corner. Initial yielding will occur at the corner with the highest moment. However, after this yielding occurs, the frame will be able to resist additional lateral shear forces with positive stiffness, before a hinge forms at the other corner, resulting in a full mechanism.

A third source of overstrength is often termed design overstrength. It relates from actions intentionally or inadvertently taken by designers. Some designers, for example, will design structures so that the demand/capacity ratios for members in frames are initially somewhat less than unity, perhaps 0.9 or 0.95. This automatically builds in overstrength. Other designers will increase the size of some members in order to limit the number of different structural shapes that are used on a project. In some cases, architects may request the use of larger members than required for structural purposes, particularly in applications employing Architectural Exposed Structural Steel (AESS). Finally, in many cases, a structure's design will be controlled by considerations other than strength to resist lateral forces. In some cases, member sizes are controlled to limit drift and in others to resist gravity demands; in either case, the structure will have greater strength than required to resist the code-specified seismic forces.

Overstrength is generally beneficial for seismic resistance in that it allows a structure to resist more intense ground shaking without onset of damage or formation of collapse mechanisms. Indeed, the seismic response modification factors specified by the code for use in calculating minimum design seismic forces presume that a minimum level of overstrength is present. However, in capacity-designed structures (see Section 3.14), overstrength can be harmful if it is not properly accounted for. For example, if a connection of a braced-frame structure is proportioned for the minimum seismic design forces, rather than for the yield strength of the braces, the connections may fail before the braces have an opportunity to yield.

The building code attempts to protect structures from the potentially harmful effects of overstrength by requiring the design of elements that are sensitive to these effects for forces that have been amplified by the overstrength coefficient, Ω_o. The overstrength coefficient represents the overstrength that can be expected of structures of different system types considering material overstrength and the typical effects of redundancy and sequential yielding. Design overstrength is only partially considered in developing these default values. Therefore, it is not always conservative in seismic design to oversize members. When substantial design overstrength is present, designers should consider calculating the overstrength by performing either a pushover analysis or a plastic analysis. The code also permits such analyses to justify the use of Ω_o coefficients less than those specified as default values.

3.16 How are design seismic forces combined with other loads?

Design seismic forces must be combined with other loads, principally dead and live loads and loads due to lateral earth pressures and fluids, in accordance with code-specified load combinations. Chapter 16 of the IBC provides three sets of load combinations:

- Strength (LRFD) load combinations;
- Basic ASD load combinations; and
- Alternative ASD load combinations.

The strength load combinations are intended for use with LRFD procedures. Either the basic or alternative ASD load combinations can be used with ASD.

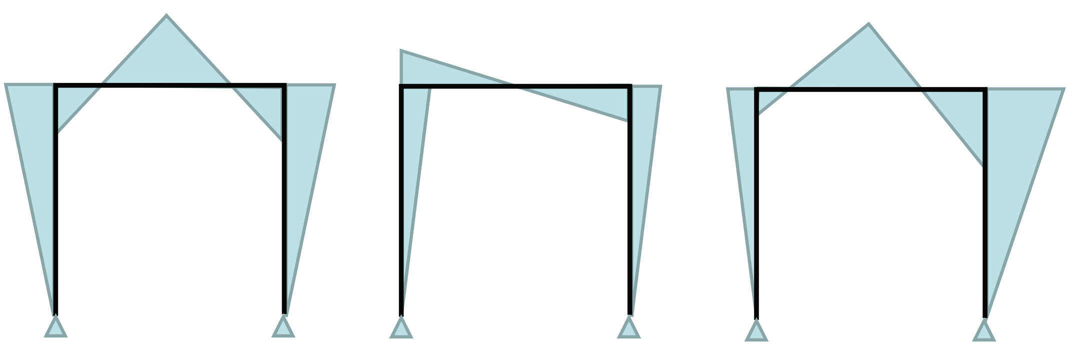


Fig. 3-2. Moment diagrams for a single-bay moment frame under various load conditions.

The primary difference between the basic and alternative ASD load combinations is that the basic combinations do not permit a one-third stress increase for load combinations incorporating transient loads, such as wind or seismic, while the alternative combinations do. The basic ASD load combinations will produce designs of steel structures that are comparable to the strength load combinations for LRFD. However, the alternative ASD load combinations may not. For some years, there has been a push by some engineers to eliminate the alternative ASD load combinations because they do not provide the same inherent protection against failure as the strength load combinations for LRFD.

SEI/ASCE 7 presents both the strength load combinations for LRFD and the basic ASD load combinations. These load combinations can be found in two different places in the standard. Chapter 2 presents the combinations applicable to all load conditions, including those that do not include consideration of earthquake forces. Section 12.4 of SEI/ASCE 7 repeats those load combinations that include seismic forces, in a somewhat clarified manner.

3.17 When should I use the AISC 341?

AISC 341 must be used in conjunction with SEI/ASCE 7 and AISC 360 for design as follows:

- For all structural steel seismic force resisting systems in Seismic Design Categories D, E and F;
- For all structural steel seismic force resisting systems in Seismic Design Categories B and C, except when using an $R = 3$ system in conformance with the requirements for a "structural steel system not specifically detailed for seismic resistance," as covered in SEI/ASCE 7, Table 12.2-1; and
- For all composite structural steel and concrete seismic load resisting systems in Seismic Design Categories B, C, D, E and F.

3.18 When should I use *R* = 3 in the design of a steel structure?

Seismic forces for buildings can be determined using an $R = 3$ system whenever the selected seismic load resisting system is designed in conformance with the requirements in SEI/ASCE 7 for a "structural steel system not specifically detailed for seismic resistance." Seismic load resisting systems in such structures must be proportioned with adequate strength and stiffness to resist design seismic forces but need not conform to any special detailing criteria or configuration limitations beyond that required by AISC 360. The code permits the use of this design approach in Seismic Design Categories B and C.

This design approach is often advantageous in that it can allow the engineer greater flexibility in configuring the structure and usually results in a lower-cost structure. There are several reasons the cost of such structures can be substantially less than those designed in conformance with AISC 341. First, structures designed in this manner can often use lighter sections. This is accomplished by avoiding the special compactness criteria of AISC 341, avoiding the need to design columns to resist axial forces amplified by the Ω_o coefficient, and avoiding the need to proportion moment frames with strong column–weak beam configurations. In addition, the connection costs of structures designed in this manner can be significantly less because connections need only be proportioned to withstand the design seismic forces, rather than to develop the expected strength of connected members.

It should be recognized, though, that structures designed in this manner may not perform as well as structures designed in conformance with AISC 341, if they actually experience a strong earthquake. $R = 3$ systems are generally not desirable for use in Occupancy Category IV facilities, where post-disaster operability is desirable. It is also important to recognize that some seismic load resisting systems covered by AISC 341 have R factors lower than 3, notably Cantilevered Column Systems and Ordinary Composite Moment Frames. If one of these structural systems is used, all of the applicable detailing requirements specified by SEI/ASCE 7 and AISC 341 should be complied with, even though a low value of R is used.

3.19 What types of steel structures can be used to provide earthquake resistance?

Seismic load resisting systems contain both vertical load-carrying elements, such as frames and walls, and horizontal load-carrying elements, including roof and floor diaphragms. The building code classifies seismic load resisting systems according to the type of vertical elements that are used to provide lateral resistance. Structural steel seismic load resisting systems generally can be categorized as braced frames, shear walls, moment frames, dual systems and cantilevered column systems.

- Braced-frame systems rely primarily on the stiffness and strength of vertical truss systems for lateral resistance. Braced frames are generally categorized as either concentric or eccentric, depending on whether the connections of braces to beams, columns and beam-to-column joints are concentric or not.

  Concentrically braced frames can have many alternative patterns, including a single diagonal brace in a bay, intersecting X-pattern braces in a bay, and inverted-V- and V-pattern braces in a bay. The latter case is also known as chevron-pattern bracing.

Buckling-restrained braced frames are a special type of concentrically braced frame with braces that are specially designed so that they can withstand yield-level compressive forces without buckling.

Eccentrically braced frames are generally arranged either as modifications of the single-diagonal pattern or chevron-pattern bracing. AISC 341 places strict limits on the eccentricities and detailing that can be used for such frames.

- Shear-wall systems rely on vertical plates, reinforced by bounding structural members, to provide lateral resistance.

- Moment-frame systems rely primarily on the rigidity of beams and columns that are interconnected in a manner that resists relative rotation between these members. Within the category of moment frames there are frames in which conventional rolled shapes are used as the beams in the frames and frames in which trusses form the horizontal members of the frames.

- Dual systems utilize a combination of moment frames and braced frames or shear walls for lateral resistance. The moment frame, acting alone, must be capable of providing at least 25% of the structure's required lateral seismic resistance, while the braced frames or shear walls that the moment frames are paired with must be proportioned, based on their stiffness, to resist that portion of the total required design lateral forces (determined considering their interaction with the moment frame, which may be more or less than 75% of the total required resistance, and may vary with height up the structure).

- Cantilevered columns systems are structures that rely on the cantilever strength and stiffness of columns that are restrained against rotation at their bases.

Each of these types of vertical seismic load resisting elements can be coupled with a variety of different horizontal elements, including wood-sheathed floors and roofs, steel deck roofs, concrete-filled steel deck floors and roofs, formed concrete slabs, precast concrete floors and roofs, and horizontal bracing systems. While the type of horizontal diaphragms that are used in a building will affect the building's stiffness, the way lateral forces are transferred throughout the structure; and the types of analyses that must be performed as part of design, it does not affect the classification of the structure's seismic load resisting system under the building code.

The building code also categorizes structural systems based on the ability of the system to undergo large inelastic deformation without loss of load-carrying capacity or collapse. The categorization related to ability to withstand inelastic deformation is generally identified through the terms Special, Intermediate and Ordinary. See Section 3.7 for additional discussion of these categorizations.

In addition to seismic load resisting systems that are composed entirely of structural steel framing, the building code also provides criteria for seismic load resisting systems that are constructed of steel and reinforced concrete systems acting compositely. Examples of these systems are braced frames that use steel beams and braces with concrete columns, moment frames that use steel beams with concrete columns, shear walls that have a reinforced concrete wall but structural steel columns and beams, braced frames and moment frames that use concrete-filled steel tubes as columns or concrete columns with structural steel cores, and wall systems where the wall is a steel plate backed by reinforced concrete.

3.20 What is the purpose of height limits and other system limitations?

The building code and SEI/ASCE 7 place restrictions on the use of some structural systems, depending on the Seismic Design Category to which the structure is assigned. The primary purpose of such restrictions is to prevent the use of systems that have limited ability to withstand strong ground shaking on sites where they are likely to experience such shaking. Generally, these limits are placed on nonductile systems that can fail in a brittle manner when subjected to loading that exceeds their elastic capacities. Examples of such systems include Ordinary Concrete Moment Frames and Plain Masonry Shear Walls, both of which are prohibited in Seismic Design Categories D, E and F. Some systems, such as steel Ordinary Moment Frames and Special Concentrically Braced Frames have limited ductility and can perform acceptably in strong ground shaking in some applications. The code permits the use of these systems in higher seismic design categories but restricts their use to structures of limited heights.

These limitations are as much a measure of the discomfort of the code development committees with the adequacy of the design requirements contained in the code as they are of any demonstrated inability of some of these systems to function well, beyond the stated limits. It is likely that these limitations will either be modified or eliminated in future editions of the code.

3.21 How do I use the IBC, SEI/ASCE 7, AISC 341, and AISC 360 together?

The IBC incorporates most structural design requirements through adoption by reference of SEI/ASCE 7, which sets the minimum required design loads, for seismic as well as other load conditions; the required combinations of the various loading conditions; and, for seismic loading, the maximum permitted interstory drift. In order to determine the required design seismic forces in SEI/ASCE 7, it is necessary to select a specific seismic load resisting system and its associated values of R, C_d and Ω_o. Once a seismic load resisting system is selected, it is necessary to detail the structure in accordance with the requirements adopted by SEI/ASCE 7 for the specific system. For seismic load resisting systems of structural steel, and those of composite steel and concrete construction, SEI/ASCE 7 generally adopts the requirements of AISC 341. The only exceptions to this are:

- Structures assigned to Seismic Design Category A; and
- $R = 3$ system (that is, a "structural steel system not specifically detailed for seismic resistance) in Seismic Design Category B and C.

For all other structures, the acceptable configurations, width-thickness ratios, lateral bracing criteria and connection design requirements are contained in AISC 341. AISC 341 defers to the requirements of AISC 360 for many of these requirements and for procedures used to determine the capacity of most structural steel and composite elements.

SECTION 4
SEISMIC SYSTEM REQUIREMENTS

4.1 What constitutes the seismic load resisting system (SLRS)?

The seismic load resisting system (SLRS) includes all of those structural elements and their connections that are essential to transferring seismic inertial forces from their point of origin to the ground. This generally includes three different element types:

- Vertical elements, which include the vertical braced frames, shear walls, moment frames and combinations of these that provide a structure's basic lateral stability.
- The foundations for the vertical elements.
- Diaphragms, which are essentially horizontal elements that transmit inertial seismic forces from their points of origin to the vertical elements of the SLRS. In essence, diaphragms act like horizontally spanning beams that distribute the inertial forces between the various vertical elements. In many steel buildings, concrete floors slabs or concrete-filled steel deck provide the necessary shear resistance for diaphragm action. In some cases, bare steel roof deck, plywood sheathing, or diagonal steel bracing—aligned in a horizontal or nearly horizontal plane—provide the necessary shear resistance for diaphragms.

In addition to these horizontal shear-resisting elements other diaphragm elements that are part of the SLRS include diaphragm chords and drag struts. Diaphragm chords act like flanges for the diaphragm and experience axial loads as the diaphragm undergoes in-plane bending. The axial force in a diaphragm chord can be calculated as the flexural moment in the diaphragm divided by the distance between chords. Drag struts are horizontal framing elements that "collect" the shear out of a diaphragm and transfer these forces, in either tension or compression, to the vertical elements. Typically beams that are in the same line as a braced frame or moment frame will act as drag struts and are part of the SLRS. Figure 4.1–illustrates these diaphragm elements for a typical steel-framed floor system.

4.2 What special requirements apply to the SLRS?

SEI/ASCE 7 specifies the minimum required strength for the SLRS in all structures, and the permissible design story drift for all structures except those in Seismic Design Category A, for which there are no specified drift requirements. The design of the SLRS for all structures must also comply with the requirements of AISC 360 and AWS D1.1. All structures

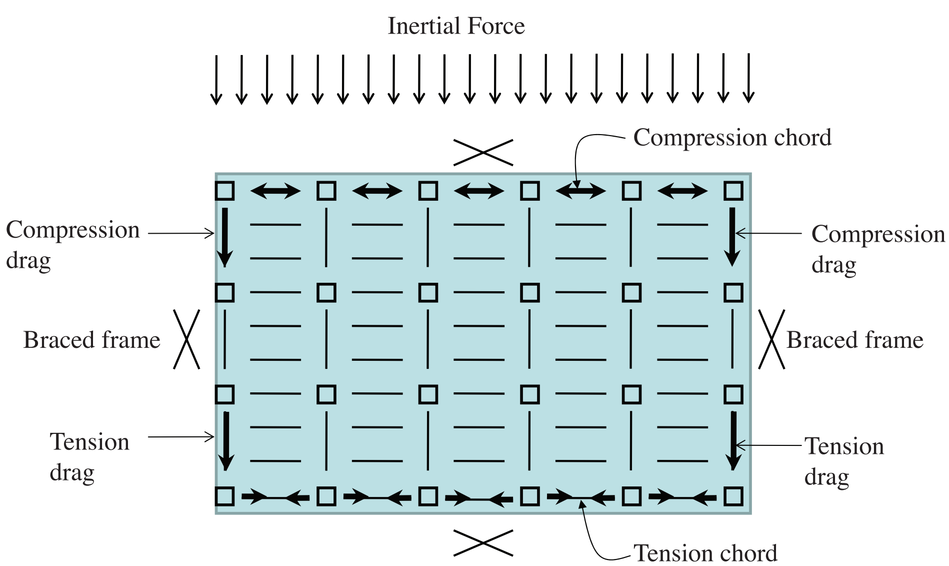


Fig. 4-1. Typical steel framing plan illustrating diaphragm drag struts and chords.

in Seismic Design Categories D, E and F, and all structures in Seismic Design Categories B and C that do not have an $R = 3$ SLRS must comply with additional design requirements of AISC 341 and AWS D1.8. These additional design requirements include:

- Limitations on the material properties of base metals and weld filler metals;
- Permissible width-thickness ratios and lateral bracing criteria for sections used in flexure and compression;
- Permissible configurations of structural systems;
- Permissible strength ratios for braces, beams and columns;
- Connection design and detailing requirements;
- Fabrication restrictions and quality assurance criteria; and
- The information that must be shown on the structural drawings.

In general, these various criteria are triggered by the selection of a specific SLRS. However, there are some requirements that apply to all SLRS. These include:

- Only members conforming to ASTM A36/36M; A53/53M; A500 Grade B or C; A501; A529/529M; A572/572M Grades 42, 50, or 55; A588/588M; A913 Grades 50, 60 or 65; A992, or A1011 HSLAS Grade 55 can be used in the SLRS. This is because materials conforming to other specifications may not have adequate ductility and toughness.
- A requirement to indicate, on the structural drawings, the members and connections that form part of the SLRS, the configuration of connections that are part of the SLRS, the welds classified as demand critical, the lowest anticipated service temperature (LAST), the location and extent of protected zones, locations where gusset plates must be detailed to accommodate inelastic rotation, and any special welding requirements.
- Filler metals used in welded connections of the SLRS must have rated Charpy V-notch toughness of at least 20 ft-lb at –20 °F.
- Bolted connections in the SLRS must be made with bolts conforming to ASTM A325 or A490, or to ASTM F1852 or F2280, which are the specifications for twist-off type bolts having similar material properties and strengths; and must be pretensioned and have surface preparation of faying surfaces with at least Class A slip resistance.
- Plates and shapes exceeding certain thickness criteria must have minimum rated Charpy V-notch toughness of 20 ft-lb at 70 °F.

4.3 What should the engineer show on the construction documents?

The IBC requires that the engineer include the design criteria for the SLRS and the quality assurance plan on the construction documents. Including the design criteria on the construction documents facilitates design review by the authority having jurisdiction, and serves as a record throughout the building's life of the basis for its design. Design criteria must include:

- The type of seismic load resisting system used;
- The Occupancy Category and Seismic Design Category;
- The Site Class;
- The values of the design spectral response acceleration coefficients, S_S and S_1;
- The values of R, C_d and Ω_o used in determining seismic design forces;
- The specifications and grades of materials used in the SLRS; and
- The foundation design values.

The quality assurance plan must include identification of the required contractor submittals and preconstruction conferences, as well as the requirements for any observations, tests and inspections that must be made during construction; identification of the party responsible for these actions; and the procedures and acceptance criteria for tests and inspections. The purpose for this information is to ensure that all parties know their responsibility in ensuring that the project conforms to the applicable quality criteria.

In addition to the requirements contained in the IBC, for structures that are designed in conformance with AISC 341, the drawings must indicate the members that are part of the SLRS, the configuration and details of connections, and the locations of demand critical welds and protected zones. The purpose for this information is to enable the fabricator and erector to understand the required details of the SLRS that only the engineer can designate.

4.4 What is a demand-critical weld?

Demand-critical welds are welds in the SLRS that may experience yield-level stresses in design earthquake shaking, the failure of which could lead to significant degradation of the SLRS and potential collapse or instability. Typically, the welded joints connecting beam flanges to columns in welded connections of Special and Intermediate Moment Frames are designated as demand critical. Welded joints in column splices and connections in Eccentrically Braced Frames may also be considered demand critical.

Filler metals used in demand-critical welds must conform to supplemental toughness criteria identified in Appendix W of AISC 341 and AWS D1.8.

4.5 What is a protected zone?

The protected zone constitutes the limited portion of certain members in the SLRS that are anticipated to undergo significant cyclic inelastic straining during response to design-level earthquake shaking. Some examples of protected zones include the anticipated region of plastic hinging in beams of Special and Intermediate Moment Frames and the links of Eccentrically Braced Frames. AISC 341 and AWS D1.8 specify special detailing and fabrication criteria within the protected zones of members of the SLRS. These criteria include a prohibition on certain types of attachments and special procedures for repairing gouges, nicks, arc strikes and similar damage that may occur during construction. The intent of these requirements is to avoid metallurgical and physical defects that could result in premature fracture and loss of strength during cyclic inelastic straining.

4.6 Why is construction quality particularly important for seismic systems?

The design of structures for seismic resistance anticipates that during design-level and more severe events, the structures will be repeatedly stressed to yield-level and more severe loadings. Under these conditions, relatively minor defects in construction can lead to the initiation of damage and failures that can compromise the performance of the entire structural system. The failures of many structures in past earthquakes have been attributed to a failure to properly implement the design requirements during building construction. Examples of common construction defects that have led to past failures in earthquakes include:

- Bracing members that are omitted, removed or cut to permit piping ducts or utilities to be routed;
- Failure to install or properly pretension bolts;
- Members that are field cut without adequate preheat or post-cut grinding; and
- Welds that are made with improper filler metals, without adequate preheat or with excessive heat input, or that contain rejectable defects.

SECTION 5
STEEL BRACED FRAMES AND SHEAR WALLS

5.1 What is a Special Concentrically Braced Frame?

A Special Concentrically Braced Frame (SCBF) is a braced-fame system in which the vertical elements of the SLRS consist of vertical trusses, composed of columns, beams and braces that conform to the configuration and detailing requirements of Part 1, Section 13, of AISC 341, as well as the general requirements of Part 1, Sections 3 through 8, of AISC 341. These systems are expected to accommodate inelastic deformation primarily through ductile tensile yielding and compressive buckling of the braces.

Acceptable configurations for SCBF are illustrated in Figure 5-1 and include frames with diagonals arranged in a single-diagonal pattern across bays, frames with diagonals arranged in an X-pattern across bays, frames with diagonals arranged either in a V or inverted-V pattern within a bay, and combinations of these patterns in different bays. Tension-only bracing is not permitted in this structural system.

Regardless of the configuration selected, the lines of action of columns, braces and beams must be concentric. Furthermore, within any single line of framing, at least 30% but not more than 70% of the total lateral force resisted by the frame at any level must be resisted by braces acting in tension, unless the compressive strength of these braces is sufficient to resist the required seismic design forces amplified by the Ω_o coefficient, in combination with other loads. This is because braces tend to lose strength rapidly when they buckle, and if too many braces are oriented such that they will all resist compressive stress at the same time, buckling of these braces could result in an unacceptable strength loss for the structure, and development of instability.

In addition to restrictions on the permissible patterns of bracing, AISC 341 also places limits on the compactness of members, the strength of connections relative to the strength of members, and the strength of some members relative to other members. Connections must be designed to resist the lesser of the expected strength in tension of the connected brace members or the maximum forces that can be delivered to the connection by the system. This limit can be determined by nonlinear analysis using expected material properties. In addition, brace connections must be configured to accommodate the end rotation of the brace as it conforms to the buckled shape, unless the connection has the strength to resist the flexural strength of the brace in the plane of buckling. This requirement is most often accommodated by allowing the gusset plate to form a flexural yield line. Finally, braces and columns in these systems must meet special compactness criteria specified in Part 1, Section 8, of AISC 341 and the beams at the apex of V and inverted-V pattern braces must be designed to accommodate the unbalanced forces that will result from buckling of either of the braces, unless the frame is configured in a "zipper" configuration. See Section 5.6 for additional information.

SCBF are often more economical than other systems designated as "special" in the building code. They combine both inherent stiffness and a moderately high value of R to provide lightweight framing solutions. The detailing of these systems is straightforward and easy to fabricate. However, many architects and building owners find the braces to be objectionable in appearance and limiting to their designs. Also, in taller structures, braced frames impose large overturning and shear forces on the foundations, which can result in massive foundations beneath the frames.

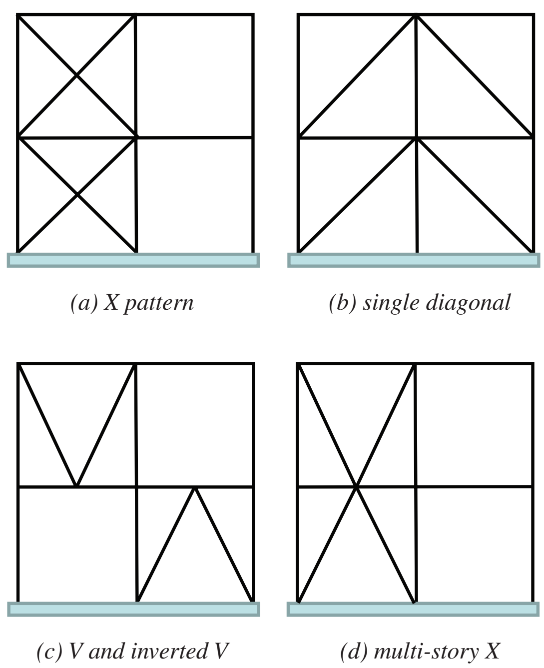


Fig. 5-1. Typical SCBF configurations.

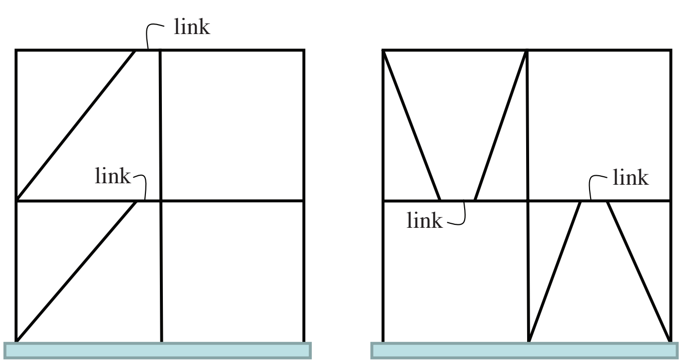
5.2 What is an Eccentrically Braced Frame?

An Eccentrically Braced Frame (EBF) is a braced-frame system in which the vertical elements of the SLRS consist of vertical trusses, composed of columns, beams and braces that conform to the configuration and detailing requirements of Part 1, Section 15, of AISC 341, as well as the general requirements of Part 1, Sections 3 through 8, of AISC 341. These systems are configured so that one end of each brace intersects a beam at a location that is eccentric to the beam-to-column joint, or eccentric with adjacent braces, such that when the frame is subjected to lateral loads, the axial forces in the braces induce flexure into the beam. These systems are expected to accommodate inelastic deformation through ductile yielding of a portion of the beam intersected by the brace, termed a link, in shear, flexure or a combination of these. In essence, the link acts as a fuse to protect the braces from buckling damage.

EBF are usually configured with single diagonals or a V- or inverted-V pattern, as illustrated in Figure 5-2.

In order to ensure that the intended ductile flexural or shear yielding of the links can occur, AISC 341 sets strict criteria for the permissible link lengths, link compactness and lateral bracing of the links. In addition, typically links must be detailed with web stiffener plates to avoid buckling of the web. Braces and brace connections must be designed strong enough to induce the desired yielding in the links. In addition, frame geometry must be controlled so that plastic rotations experienced by the link or link-to-column connections do not exceed acceptable values.

EBF are a relatively economical high-performance structural system. They tend to be stiffer than moment frames and, therefore, experience reduced levels of lateral drift, which can help to protect nonstructural elements, such as cladding. Although they have a higher *R* value than concentrically braced frame systems and, therefore, can be designed for smaller seismic forces than concentric systems, the design and detailing requirements for the link beams often result in increased steel tonnage and more expensive fabrication and erection. These systems do perform better than concentrically braced frames and are a better choice for Occupancy Category III and IV structures than concentrically braced frames. Also, some architects find that the eccentric location of the brace-to-link beam connections facilitates their placement of windows, doors, and other architectural elements.



(a) single diagonal *(b) V and inverted V*

Fig. 5-2. Typical configurations of EBF.

5.3 What is a Buckling-Restrained Braced Frame?

A Buckling-Restrained Braced Frame (BRBF) is a special type of concentrically braced frame in which the braces are detailed such that inelastic deformation of the structure can be accommodated primarily through ductile yielding, both in compression and tension of the braces. This is typically accomplished by detailing the brace with a ductile core that is intended to carry the axial forces induced by lateral loading, encased and restrained within a surrounding tube. The tube does not actually participate in resisting the brace forces but, instead, braces the core against buckling when acting in compression. The braces must be constructed in such a manner that the outer tube can restrain the core against buckling while allowing it to elongate and shorten independent of the tube. While the concept of a buckling-restrained brace is not by itself proprietary, the detailing and quality assurance measures required by AISC 341 for this system are such that braces are usually purchased from one of several suppliers who have proprietary brace systems that meet the AISC criteria.

BRBF are considered a high-performance structural system because the braces can undergo substantial yielding without incurring noticeable damage. BRBF configurations are typically limited to single-diagonal or V- or inverted-V patterns, similar to those shown in Figure 5-1.

BRBF couple the economy of concentrically braced frames with the high-performance capability of eccentrically braced frames. This system has recently become one of the most popular systems for use in Occupancy Category III and IV structures in higher Seismic Design Categories.

5.4 What is a Special Plate Shear Wall?

A Special Plate Shear Wall (SPSW) is a structural system in which the vertical elements of the SLRS consist of steel frames stiffened by thin steel plate walls. Inelastic deformation of the structure is intended to occur through the development of diagonal tension-field action in the web of the steel plate. Design and detailing requirements for this system are contained in Section 17 of AISC 341.

Figure 5-3 illustrates a typical SPSW. As shown in the figure, the columns and beams that frame the shear panel are termed vertical boundary elements and horizontal boundary elements, respectively. These elements are designed to distribute the plate tension forces between panels, resist the plate tension forces at the perimeter of the plate panel system, resist the concentrated overturning forces that occur at the edges of the wall, and provide out-of-plane stability for the wall. AISC 341 requires that detailing of connections of horizontal boundary elements to vertical boundary elements must comply with the requirements for fully restrained moment connections, though OMF moment connections are permitted because system drifts are limited and the joint rotations expected are only modest. The plate is attached to the boundary elements with welds capable of developing the plate strength.

SPSW are highly ductile and may offer an attractive design solution for buildings in which the placement of structural walls around service cores for elevators, stairwells and utility chases can provide adequate seismic resistance. The thin plate of the wall makes it relatively simple to conceal these structural elements within the building's architecture. However, like braced frames, shear walls produce large overturning forces on foundations. In addition, the extensive field welding required for this system can result in relatively high construction costs.

5.5 What is an Ordinary Concentrically Braced Frame?

An Ordinary Concentrically Braced Frame (OCBF) is a braced-frame system similar to the SCBF, but with less stringent detailing criteria and less restriction on the permissible configuration. In addition to the frame configurations illustrated in Figure 5-1, OCBF can be configured with K-pattern bracing as illustrated in Figure 5-4.

The K-pattern bracing configuration can have undesirable inelastic behavior characteristics as once one of the braces in a story buckles in compression, the remaining brace will place a large unbalanced load on the column, which can cause the column to yield at mid-height and possibly buckle.

The design and detailing requirements for OCBF are contained in Chapter 14 of AISC 341. These are relatively simple and consist of a limitation on the permissible slenderness of the braces and a requirement that connections be designed for the axial strength of the brace, or the forces associated with load combinations containing seismic forces amplified by the Ω_o coefficient. In addition, in V-, inverted-V- and K-pattern configurations, the beams and columns at the apex of the braces must be designed for the unbalanced forces anticipated following buckling of the compression brace and yielding of the tensile brace. Also, in V-, inverted-V- and K-pattern configurations, the braces cannot be relied upon to support dead or live loads.

OCBF can be more economical than SCBF because of the relaxed detailing criteria. However, they are also more likely to experience severe damage during design or higher intensity shaking than SCBF. As a result, the building code assigns a relatively low value of *R* to this system, and also limits its use in Seismic Design Categories D, E and F. One of the permitted uses of this system in higher seismic design categories is in seismically isolated structures. The code permits its use in such structures because the presence of the isolators should effectively shield the SLRS from experiencing significant overload and damage.

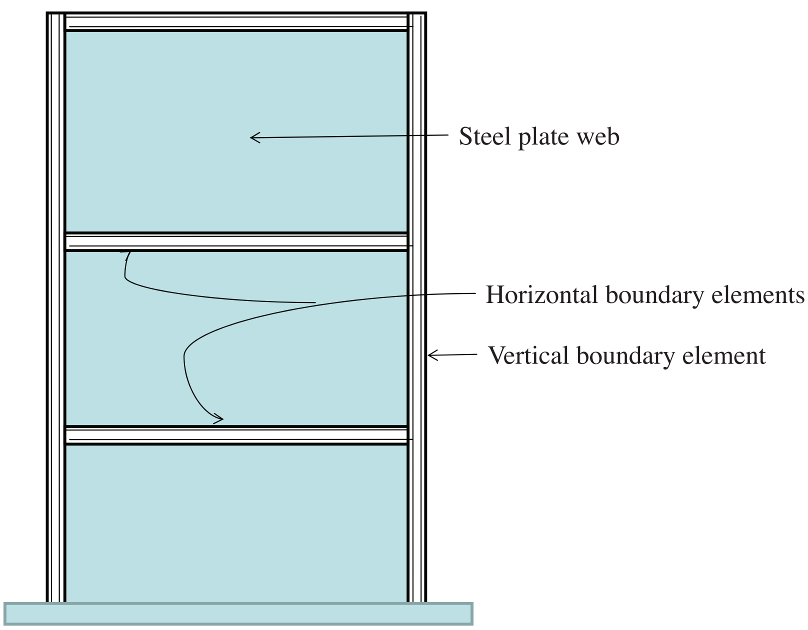


Fig. 5-3. Special plate shear wall.

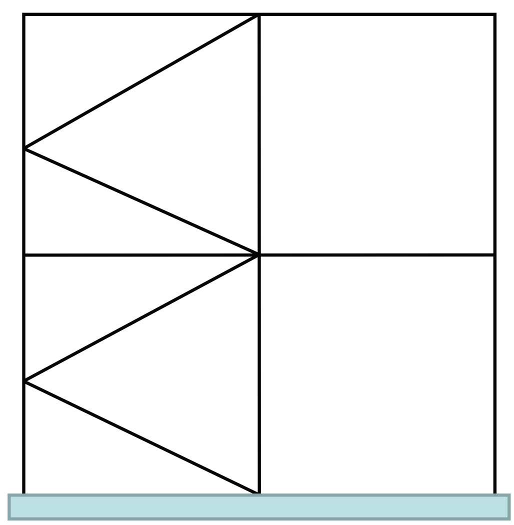


Fig. 5-4. K-pattern braced frame configuration (permitted for OCBF only).

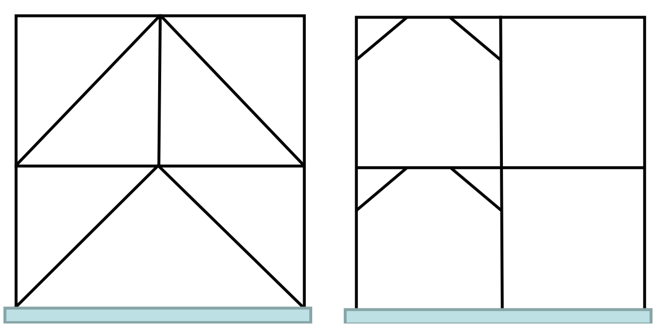
5.6 What are the common braced-frame configurations?

Figures 5-1, 5-2 and 5-4 illustrate the most common braced-frame configurations. In addition to these, a number of other configurations are also possible. Figure 5-5 illustrates several of these, including a zipper configuration and knee-bracing. The zipper-configuration braced frame is a form of SCBF that is similar to V-pattern bracing, but it behaves somewhat better in strong earthquake shaking. After the compression brace in the lower story of this frame buckles, the "zipper column" that extends between the apex of the lower and upper bracing system prevents the beam at the apex of the lower system from being deformed downward by the tensile brace as the compression brace loses strength. This enables the frame to develop more gradual yielding and to dissipate more energy before forming a mechanism.

A knee-braced frame, which once was frequently used in steel industrial buildings, does not qualify as a concentrically braced frame and is not permitted in either SCBF or OCBF. Rather, it is a hybrid between a braced frame and a moment frame as the braces form a rigid beam-to-column joint, which then permits lateral forces to be resisted through flexure of the beams and columns. It can be used in $R = 3$ systems and also as an OMF (see Section 6.5) as discussed in the Commentary to AISC 341.

A two-story X-pattern braced frame incorporates some of the architectural advantages of V- and inverted-V-pattern braced frames but eliminates the requirement to design the beam at the apex of the braces for the unbalanced forces resulting from compression buckling of the braces. This configuration can be used in either SCBF or OCBF.

Another bracing system configuration that was once very popular in industrial construction is an X-pattern bracing configuration, like that illustrated in Figure 5-1, in which the braces are intentionally selected as very slender members, often using rods or single angles. These frames were designed as tension-only systems, neglecting the small compressive capacity of these slender braces and proportioning the braces so that they could resist 100% of the required lateral forces through tensile behavior. This is a very economical system but one that does not perform well in earthquakes that produce inelastic deformation in the structure. Because such frames cycle back and forth under reversed ground motion, shock forces are induced in the braces as they rapidly transition from a buckled state under compression to a tensile state. This shock can fracture both braces and connections, resulting in a loss of lateral stability. Accordingly, tension-only bracing systems are prohibited in SCBF; for OCBF, they are permitted in Seismic Design Categories B and C, limited in Seismic Design Categories D and E, and prohibited in Seismic Design Category F.



(a) zipper frame *(b) knee-braced frame*

Fig. 5-5. Zipper frame and knee-braced frame configurations.

5.7 What is a staggered truss system?

A staggered truss system is a form of concentrically braced frame in which the braced frames act both as vertical load-carrying trusses in multi-story structures and lateral bracing elements. As illustrated in Figure 5-6, each frame consists of alternating stories in which there is either a full-story depth vertical truss or an open bay. Often, the central panel of the trusses has no diagonals, relying on Vierendeel behavior of the truss chords for stability. Two configurations of frames are used together: one in which the story-high trusses are present at even stories and one in which they are present in odd stories, with each of these frame types alternated along the length of the building. This style of construction can be advantageous in multi-story residential buildings, where the trusses can be placed in party walls between units. It can provide a very economical solution in that there no interior columns, and the beams framing between the columns can be lightweight, because they are supported by the trusses in which they serve as chords.

This system has a vertical discontinuity irregularity and also places large demands on the diaphragms, since the floor slabs at each level must transfer lateral forces between adjacent frames. As a result, it has not yet been demonstrated to be a practical design solution for the highest seismic design categories. However, it has been advantageously used in Seismic Design Category A and as an $R = 3$ system in Seismic Design Categories B and C.

5.8 What should the designer be aware of about braced-frame systems?

Although a braced frame can be a very economical system, they have a number of characteristics of which the designer should be aware. The most obvious is that the presence of the braces can restrict views and block horizontal access and transport across lines of framing, complicating architectural design. Many owners and architects find these features objectionable, unless the braced frames can be hidden adjacent to property line walls or building core walls. Early discussions with the architect and owner can help to mitigate these concerns.

Braced-frame systems usually produce large foundation overturning loads that can be a challenge in design. In particular, the design of column connections to foundations to transfer both the large shear and uplift forces that can simultaneously occur can be quite challenging. Often it is necessary to employ piles, caissons, soil anchors or other deep foundation types to resist these large overturning forces. In taller buildings, the use of outriggers and hat trusses can help to mitigate these effects somewhat.

The gusset plates at brace connections, particularly in SCBF, can also be quite challenging. Sometimes, in order to develop the required design forces, gusset plates must become so large that braced bays appear to be totally occupied by braces and gussets, almost to the extent that a nearly solid wall of steel is present (see Figure 5-7). Particularly in areas of the United States where the engineer delegates detailed connection design work to the contractor, engineers should form a good understanding of the likely size and shape of gusset plates before completing their construction documents.

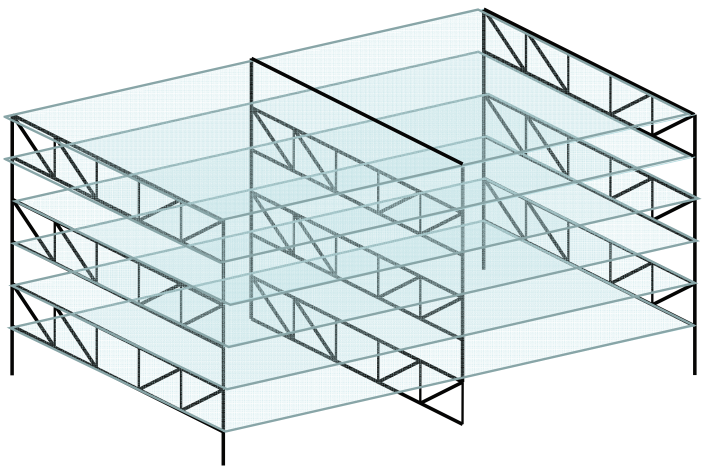


Fig. 5-6. Staggered truss system.

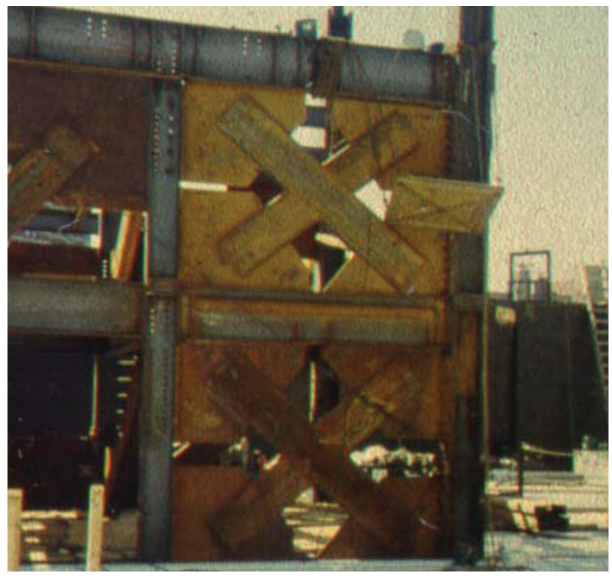


Fig. 5-7. Large braces and gusset plates occupying almost the entire bay. (Photo courtesy of Cives Steel Company)

SECTION 6
STEEL MOMENT FRAMES

6.1 What is the difference between fully restrained and partially restrained moment connections and full-strength and partial-strength moment connections?

Moment frames derive their strength and stiffness through the section properties of the beams and columns that form the frame and the interconnection of these elements at beam-to-column connections. Beam-to-column connections in moment frames can generally be categorized as either full or partial strength and either fully or partially restrained.

A fully restrained connection is capable of holding the angle between beams and columns essentially constant throughout the range of elastic behavior. Very few connections are actually fully restrained because, under loading, some deformation will invariably occur in the connection's panel zone and possibly in other locations, such as connection plates and bolts. AISC 360 is silent as to how much flexibility such elements can have before they prevent a connection from being classified as fully restrained. FEMA 350 suggests that if the flexibility of the connection elements—excluding panel zones—exceeds 10% of the beam flexibility, the connection should not be considered fully restrained. If a connection does not qualify as fully restrained, it is classified as partially restrained. By definition, partially restrained connections permit the angle between connected beams and columns to vary as loads below the elastic limit are applied to the assembly.

Full-strength connections are capable of developing the yield strength of the weaker of the connected columns and beams before significant yielding occurs in the elements of the connection. Such connections will permit plastic hinges to form in either the beams or columns when the joint is loaded in rotation. Partial-strength connections will experience yielding of some connection elements before plastic hinging can form in the connected beams or columns.

Most all-welded moment connections are considered to be both fully restrained and full strength. While it is possible to design bolted connections that can develop the full strength of the connected elements, these connections tend to be more flexible than their welded counterparts. Nevertheless, some bolted connections do qualify as both full strength and fully restrained. Regardless of whether they are bolted or welded, in reality many connections that are deemed to be fully restrained have some flexibility and will also experience some yielding within their panel zones, prior to formation of full plastic hinging of the connected members.

Connection rigidity is an important concern in the design of moment frames for seismic resistance. This is because moment frames are inherently flexible systems, and the design of moment frames for seismic resistance is commonly controlled by the building code's requirements to limit interstory drift under seismic loading, rather than the requirements to provide minimum strength. If beam-to-column moment connections have significant flexibility, this tends to make the frame as a whole more flexible and requires the use of larger members to control drift to specified levels than would otherwise be required. Nevertheless, in lower seismic design categories, economical structures can be designed that use relatively low-cost, partially restrained moment connections at all beam-to-column connections, mobilizing the entire building frame in seismic resistance.

Connection strength is also an important consideration for seismic design. If inelastic frame deformation occurs through yielding of connections, rather than the beams and columns they connect, large concentrated ductility demands may occur in these connections. It is generally preferable to accommodate the inelastic demands on a frame through distributed yielding of the connected members. Nevertheless, some partial-strength connection details have been demonstrated to provide sufficient ductility for service in SLRS. An advantage of such connections is that it is often easier to repair damaged connection elements, after an earthquake, than it is to repair yielded or buckled beam and column flanges, which is a common occurrence when plastic hinges form in beams and columns.

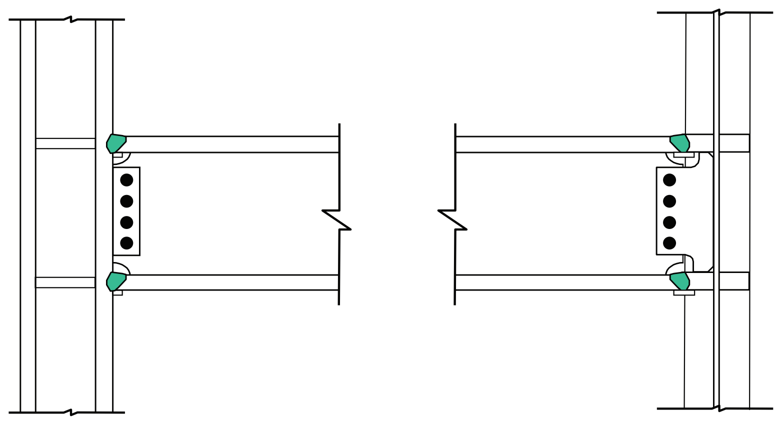
6.2 What is a prequalified connection?

Prior to the 1994 Northridge earthquake, the building code defined two types of steel moment frames: an ordinary moment frame (OMF) and a special moment-resisting frame (SMRF). In higher seismic design categories, the OMF was limited to use in buildings with a height of 160 ft or less. For the SMRF, the building code prescribed the use of a single detail that was a less-well-detailed version of what today is called the welded unreinforced flange–bolted web connection. The engineer could use an alternative detail if he or she could demonstrate that such a detail were capable of providing adequate ductility when loaded inelastically.

The prescriptive pre-Northridge moment connection, illustrated in Figure 6-1, consisted of a bolted single-plate shear connection between the beam web and column and complete-joint-penetration (CJP) groove welds between the beam flanges and column. Based on limited testing at the University of California at Berkeley during the 1960s and 1970s, this connection was considered to be full strength, fully restrained and highly ductile. Design of this connection type generally assumed that the bolted web connection would transfer 100% of the beam shear to the column, and none of the flexure, and that the welded joints of the beam to the column would transfer 100% of the beam's plastic moment capacity.

Shortly following the 1994 Northridge earthquake, engineers discovered that a number of these prescriptive connections had experienced brittle fractures under relatively low levels of seismic loading. Fractures generally initiated at or near the CJP groove weld of the lower beam flange to column flange joints. Figures 6-2 through 6-4 illustrate several of the common patterns of fracture that were observed including fractures that extended through the bottom beam flange (Figure 6-2), fractures that extended into the column flange and permitted a divot of steel to be pulled out of the column flange (Figure 6-3), and fractures that extended completely through the column flange and into the column web (Figure 6-4).

The discovery of these fractures in a number of buildings caused great dismay among engineers and building officials. The International Conference of Building Officials (ICBO), a predecessor organization to ICC, adopted an emergency change to its Uniform Building Code (UBC) that removed the prescriptive connection from the code and instead required that engineers demonstrate, through a program of qualification testing, that connections used in a building are capable of adequate inelastic cyclic performance. AISC 341 adopted similar requirements.



(a) strong-axis connection *(b) weak-axis connection*

Fig. 6-1. Typical prescriptive pre-1994 moment connections.

Fig. 6-2. Fracture of beam flange at CJP groove weld of beam flange to column flange. (Photo by Dave Norris)

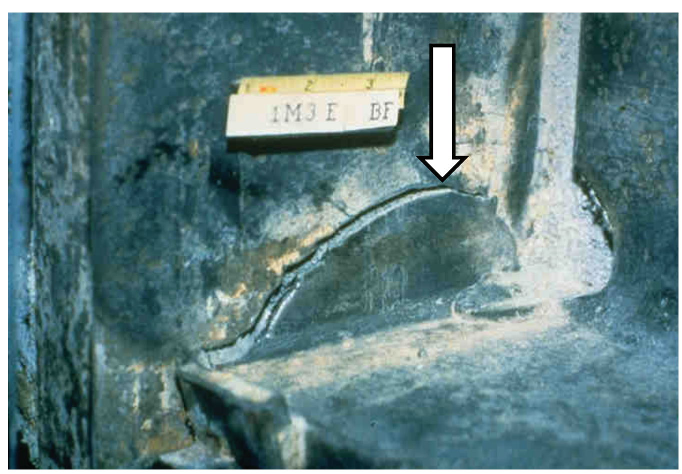


Fig. 6-3. Fracture involving divot of steel being withdrawn from column flange. (Photo by Dave Norris)

This performance requirement essentially imposed on each project the need to do full-scale laboratory testing of connections—a costly endeavor—or to use a connection design that had been tested by someone else, assuming the engineer could obtain the permission of the person who developed the connection detail. Since no documentation of suitable connection testing was publicly available, engineers needed guidance on how to proceed.

Following the adoption of the emergency code change, FEMA funded the SAC Joint Venture, a consortium of three organizations—the Structural Engineers Association of California (SEAOC), the Applied Technology Council (ATC), and the California Universities for Research in Earthquake Engineering (CUREe)—to perform a program of research and develop practice guidelines that would provide engineers with reliable design approaches for steel moment-frame structures. The six-year project culminated in August 2000 with the publication of the FEMA 350, 351, 352, 353 and 355 reports.

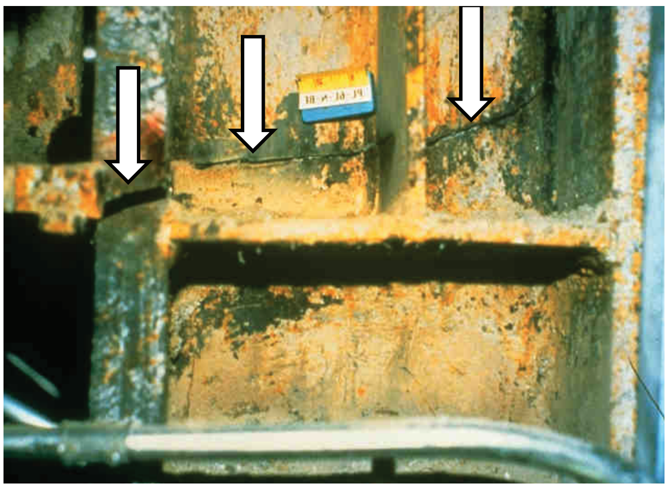


Fig. 6-4. Fracture extending through column flange and web. (Photo by Dave Norris)

FEMA 350 included a series of connections that had been developed, tested and demonstrated capable of providing adequate cyclic inelastic behavior, if properly designed and constructed. These connection types were designated as "prequalified" in FEMA 350, indicating that further qualification testing on a project-specific basis would not be required for these connections, if they were used with the limits specified in FEMA 350.

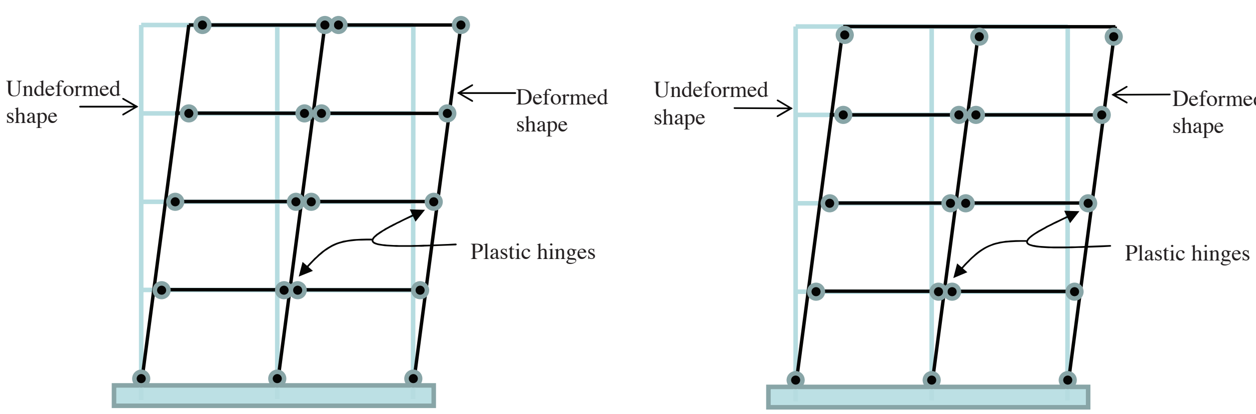
Based upon this work, AISC 341 later adopted the concept of prequalified connections into its provisions. Under AISC 341, a prequalified connection is any connection that has been approved by an appropriate connection prequalification review panel to be capable of meeting the performance criteria of AISC 341, under specified limitations. AISC then established such a review panel, which has developed AISC 358. This standard, which is continuously updated and improved, contains a series of prequalified connection types and associated details, fabrication requirements and applicability limits. In addition, several proprietary connections have been prequalified through alternative approval processes.

6.3 What is a Special Moment Frame?

A Special Moment Frame (SMF) is a moment frame that meets the configuration and detailing criteria contained in AISC 341, Section 9. To qualify as an SMF, the frame must comply with the following:

- Beams and columns must be seismically compact, and must be laterally braced at sufficient intervals to ensure that flexural yielding can occur.

- Connections must be full strength, or approximately so.



(a) beams and column bases hinge　　*(b) beams, column bases and tops hinge*

Fig. 6-5. Anticipated sidesway mechanisms in SMF.

- Columns must have sufficient flexural strength to satisfy strong column–weak beam criteria (see Section 6.7).

- Column webs must have sufficient strength in the panel zone (the area of column web between attached beam flanges) to develop the expected flexural strength of the beams.

- Column splices must be capable of developing the strength of the column in flexure and shear.

- Connections must be demonstrated, in accordance with Appendix S of AISC 341 to be capable of sustaining at least 0.04 radian of interstory drift demand without degradation of beam flexural strength below 80% of the nominal plastic moment capacity of the beam.

Frames that are configured in this manner should be capable of developing plastic mechanisms consisting of the formation of plastic hinges in the beams, near the beam-to-column joints and at the bases of columns, as illustrated in Figure 6-5a, or at the tops of columns, as illustrated in Figure 6-5b. Such frames should be capable of sustaining large inelastic drift demands without failure or loss of significant strength. This is because the inelastic drift should be uniformly distributed up the structure's height, and inelastic deformations are concentrated in compact sections that are adequately braced to withstand these deformations without lateral-torsional buckling of the section and without excessive strength loss.

These frames are permitted to be designed using very large values of the seismic response modification factor, *R*. As a result, the selection of member sizes in these frames is usually controlled by consideration of limiting interstory drift.

6.4 What is an Intermediate Moment Frame?

An Intermediate Moment Frame (IMF) is a moment frame that meets the criteria of AISC 341, Section 10. Beams and columns must be seismically compact and connections must be capable of resisting at least 0.02 radian of interstory drift demand, in accordance with the criteria of Appendix S of AISC 341. However, there are no specific strong column–weak beam requirements. In addition, panel zones need only be strong enough to develop the design seismic forces, in combination with other loads as required by the building code, rather than being able to develop the strength of the beam.

IMF may not form the sidesway mechanisms illustrated in Figure 6-5 and do not have the capability to reliably withstand the large inelastic deformations of SMF. As a result, the permissible seismic response modification factor, *R*, for these systems is lower than that for the SMF. Still, the design of these frames is often controlled by considerations of drift. Because IMF have reduced ability to reliably withstand large cyclic inelastic deformation as compared to SMF, the building codes limit the use of IMF in some applications in higher seismic design categories.

6.5 What is an Ordinary Moment Frame?

An Ordinary Moment Frame (OMF) is a moment frame that meets the requirements of Section 11 of AISC 341. OMF are not anticipated to undergo significant inelastic deformation. As a result, there are relatively few limitations on the configuration of these structures or their connections. Connections may be designed as either partially or fully restrained. Partially restrained connections can also be designed as partial-strength. Fully restrained connections are required to have adequate strength to develop the expected plastic moment capacity of the weaker of the connected beams or columns, so that if yielding of the frame occurs, it is accommodated in the members, rather than within the connection. In addition, if connections are made using CJP groove welds of beam flanges to columns, weld access holes must conform to prescriptive geometry requirements contained in AISC 341, and backing bars, if used, must be removed from the bottom flanges of beam to column connections.

Since OMF are not required to be designed or detailed to withstand significant inelastic deformation, the building code requires the use of a relatively low value of the seismic response modification factor, *R*. As a result, design of these frames may be controlled by considerations of strength, rather than drift control. Because the ductility of OMF is quite limited, the building codes limit their use in zones of high seismic activity to lightly loaded structures.

6.6 What is a Special Truss Moment Frame?

A Special Truss Moment Frame (STMF) is a type of moment frame in which the horizontal framing members are trusses that have been specially detailed to accommodate inelastic deformation through yielding of the panels in the center of the truss span, forming a yield mechanism like that illustrated in Figure 6-6. In order to ensure this type of behavior, the trusses must be specially designed and detailed such that the diagonals in the central panel can yield and buckle in a ductile manner, and the truss chords can form plastic hinges at the ends of the panels without loss of strength. The chords must meet special compactness and lateral bracing requirements. The chords, and diagonals other than those in the link panels, must have sufficient strength to withstand the shear and bending forces resulting from gravity loads, together with seismic shears associated with development of full yield in the link panels, considering both the axial yielding/buckling of the braces in the panel and development of plastic moments in the chords. Section 12 of AISC 341, under which these structures are designed, also permits the central panel to be constructed as a Vierendeel, without braces.

STMF can be an attractive solution for applications in which relatively long spans (up to 65 ft) are required. They will be most economical when used in buildings with a geometry that permits multiple trusses to have the same geometry and section properties, so that repetitive fabrication techniques can be used.

6.7 What is a strong column–weak beam condition?

A strong column–weak beam condition is a configuration in which the columns of a moment frame are proportioned with sufficient flexural strength such that under lateral loading, inelastic behavior of the frame will be controlled by flexural yielding of the beams, rather than the columns. This condition enables development of the types of sidesway mechanisms illustrated in Figure 6-5 and prevents, in an indirect manner, the formation of a story mechanism. Sidesway mechanisms like those shown in Figure 6-5 enable the frame to dissipate the maximum amount of earthquake energy, during inelastic cycling, and also result in the highest overstrength and redundancy. AISC 341 requires proportioning of frames to satisfy strong column-weak beam conditions in SMF, except at the roof level.

The amount of flexural strength required of columns in order to ensure the development of sidesway mechanisms, rather than single-story mechanisms, is dependent on the axial load in the columns, which reduces their available flexural strength, the variation in possible yield strength of the beams, the amount of strain hardening that may occur in the beams, and the total amount of lateral deformation in the frame. Recent research has shown that the requirements for strong column–weak beam conditions contained in AISC 341 may not be adequate to ensure that sidesway mechanisms can develop in frames when the frame supports large masses and undergoes significant lateral deformation because P-Δ effects can significantly alter the distribution of moment demands on the frame elements.

Another method of ensuring that sidesway mechanisms can develop is to design frames such that the panel zones (the portion of the column web located between the upper and lower flanges of the attached beams) will yield in shear before either the beams or columns can develop their full plastic moments. Figure 6-7 illustrates how shear develops in the panel zone of a beam-to-column moment connection.

As shown in this figure, the panel zone is defined by the portion of the column web between the beam flanges, having depth, d_b, and width, d_c. The moment in the beam, M_b, is resolved into a couple of forces, F. The shear force across the panel zone, V_p, is given by Equation 6-1.

$$V_p = \frac{F - V_c}{d_c t_{cw}} \tag{6-1}$$

In this equation, V_c is the story shear in the column, which always opposes the panel zone shear, and t_{cw} is the thickness of the column web. The other parameters are illustrated in Figure 6-7.

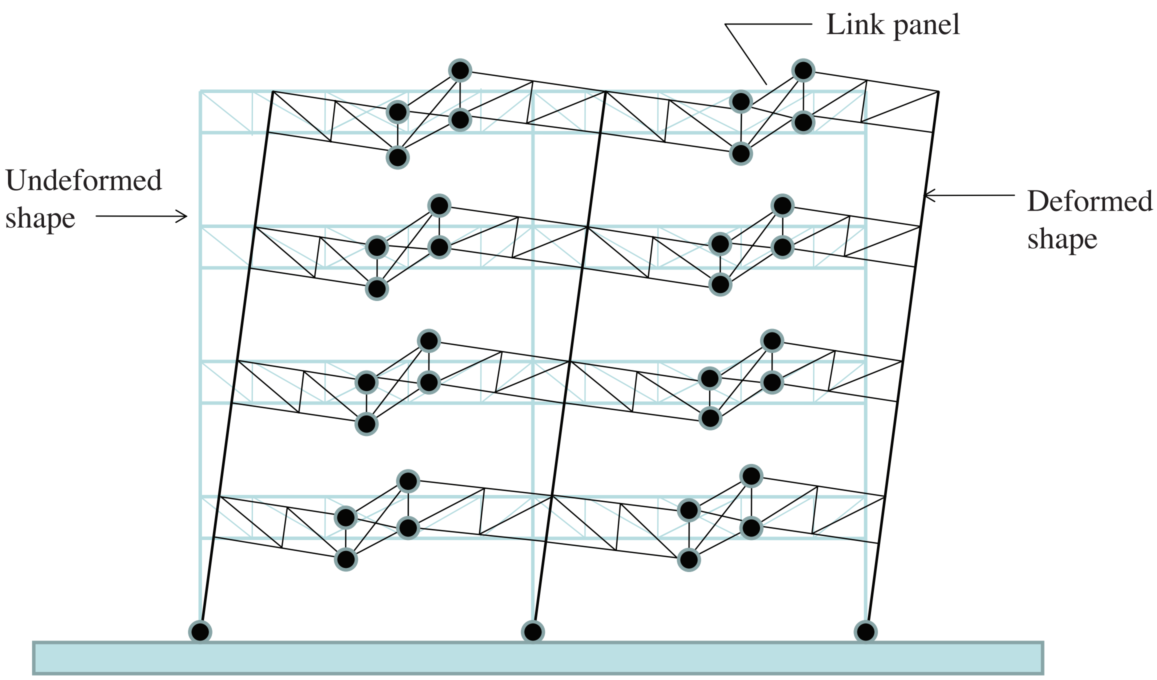


Fig. 6-6. Deformed shape of STMF.

If the shear strength of the panel zone is sufficiently small, the forces in the beams and columns will be limited by yielding in the column web. If a frame is controlled by this type of behavior, the sidesway mechanism illustrated in Figure 6-8 will occur. This results in the formation of fewer plastic hinges than the mechanism in which beam yielding occurs, illustrated in Figure 6-5; however, it is capable of dissipating significant amounts of energy as well as providing high overstrength and redundancy. AISC 341 discourages designs in which panel zones are the primary zone of yielding because excessive shear deformation of the panel zone has been found to lead to premature fracturing of the beam flange to column flange welded joint. However, as noted previously, weak panel zones are permitted at the roof level and also in IMF and OMF.

6.8 How can panel zones be modeled?

Even if panel zones are strong enough to force primary inelastic behavior to occur in the beams in a moment frame, significant elastic deformation can occur in these panel zones before and as beam hinging occurs. This shear deformation contributes to overall frame drift under lateral load. It is important to capture this effect when performing seismic analysis of moment frames so that the drift is not underpredicted.

There are a number of ways to model panel zones to account for their deformations. These range from simple approaches that extend the effective length of the beams and columns into the panel zones to more complex approaches that use scissor-type elements to represent the panel-zone deformation characteristics. A report published by the National Institute of Standards and Technology (NIST) on steel SMF (Hamburger et al., 2009) provides guidance on the use of these techniques. However, these sophisticated models are seldom necessary for design purposes.

During the studies conducted by the SAC Joint Venture, following the Northridge earthquake, researchers determined that using a column centerline-to-column centerline length for beams approximated the flexibility of panel zones well for many moment frames. Many designers assume that the panel zone provides some stiffening of the beams, and use an effective rigid panel zone that has dimensions roughly half that of the actual panel zone. That is, the beam and column lengths are extended by one-quarter of the corresponding panel-zone dimension in the model at each end. While this is a common procedure, it often results in a model that is too stiff and that will underestimate frame drift.

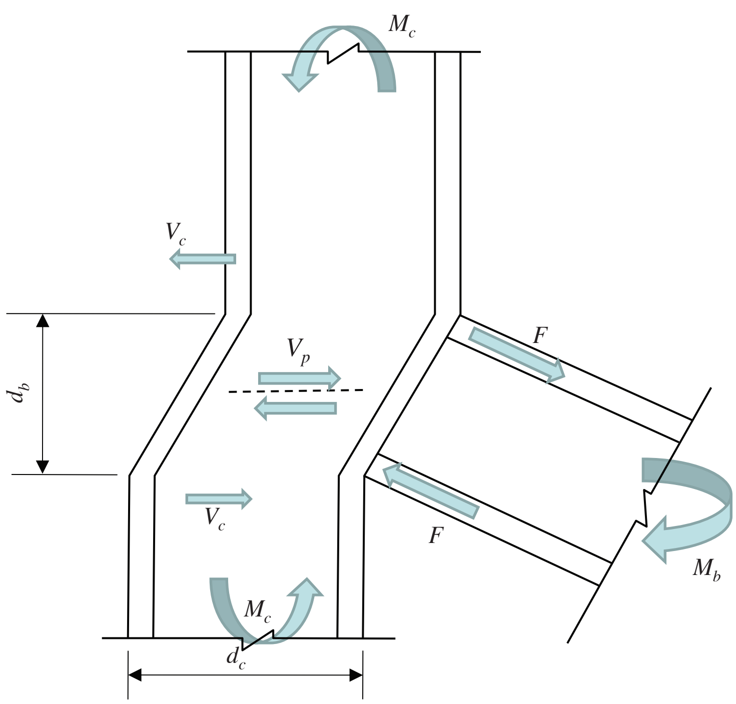


Fig. 6-7. Panel-zone shear in columns.

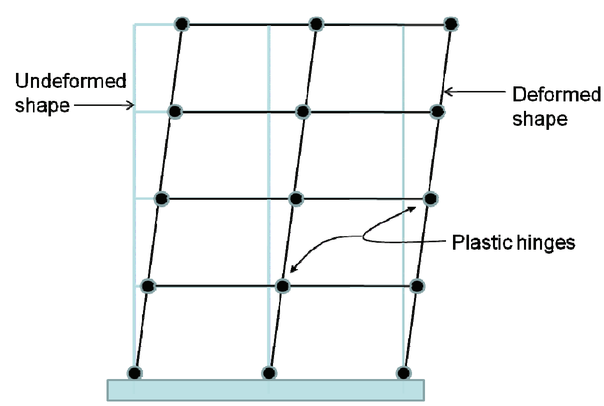


Fig. 6-8. Sidesway mechanism controlled by panel-zone yielding.

SECTION 7
DUAL SYSTEMS

7.1 What is a dual system?

A dual system is a type of SLRS that uses a combination of moment frames and braced frames or shear walls as the vertical elements. When this system is used, the various vertical elements must be proportioned to resist the design seismic forces based on their relative rigidities, as determined by structural analysis. However, regardless of the forces that the analysis indicates the moment frames will resist, the moment frames must be proportioned with sufficient strength to resist at least 25% of the total specified seismic design forces for the structure, without consideration of the braced frames or shear walls.

In essence, the moment frame is considered to be a backup redundant system for the braced frames or shear walls, which typically are proportioned to carry most of the seismic design forces. In theory, the presence of this redundant system will enable the structure to survive intense earthquake shaking that extensively damages the primary system.

SEI/ASCE 7 recognizes several types of dual structural steel systems, including SMF combined with EBF, SCBF, BRBF, or SPSW, and IMF combined with SCBF. Each of the types of frames in a dual system structure must comply with the applicable requirements of AISC 360 and AISC 341.

7.2 What are the advantages of a dual system?

In Seismic Design Categories D, E and F, the building code places restrictions on the height of structures with braced frames or shear walls alone. However, dual systems with SMF are not restricted in height. Thus, by providing a moment frame system capable of resisting at least 25% of the required seismic design forces, it is possible to use braced frame or shear wall systems in very tall structures. In addition, the building code permits the use of higher seismic response modification factors, *R*, for dual systems than are otherwise permitted for braced frames and shear walls.

In tall structures, the braced frames or shear walls and moment frames tend to interact such that the braced frames or walls carry most of the seismic forces at lower levels and the moment frames carry much of the forces at upper levels. This tends to reduce overturning demands on the braced frames and walls while limiting the deflections of the moment frames, resulting in very economical structural solutions for tall buildings.

SECTION 8
CANTILEVERED COLUMN SYSTEMS

8.1 What is a cantilevered column system?

A cantilevered column system is a type of SLRS in which the vertical elements of the system consist of individual columns that are cantilevered from their bases. Steel cantilevered columns have been commonly used in the first story of residential structures, such as with the column placed behind the short sections of wall around garage door openings. In this case, the foundations are designed to resist the overturning moments from wind and seismic loads at the column bases. Cantilevered column systems are sometimes also used in the top story of multi-story commercial structures. In this application, the extension of the column through the story below the top story is used as a back-span to resist base moments.

8.2 What is the difference between the several steel cantilevered columns systems?

SEI/ASCE 7 defines three different types of steel cantilevered column seismic force resisting systems. These are cantilevered columns detailed to the requirements for SMF, IMF and OMF, and the intent of SEI/ASCE 7 is that the columns used in these systems must meet the corresponding section compactness requirements specified by AISC 341 for each system.

It could also be inferred that the base connection of cantilevered columns must conform to the criteria of AISC 341 for these moment-frame systems. However, in reality, there are no prequalified base connections in AISC 358. Regardless, designers using these systems should adapt appropriate prequalified beam-to-column connections for use in these applications, substituting the base plate for the column flange in these details.

Recently, the AISC Committee on Specifications has questioned the need for three different types of cantilevered column framing systems and has initiated action to replace the three current systems with a single system having only one set of detailing requirements.

8.3 Why are the *R*-factors for cantilevered column systems so low?

Cantilevered column systems have several attributes that make them undesirable for use in resisting intense earthquake shaking unless they are designed to remain essentially elastic during this shaking. One of these is that the system does not provide for sequential yielding before forming a sidesway mechanism, resulting in inherently low overstrength as compared with other systems. Another is that the system is inherently flexible and will experience large story drifts, particularly if the bases are not provided with adequate rigidity. Under the resulting large drifts, P-Δ instability can occur, resulting in premature failure.

Prior to the 1994 Northridge earthquake, cantilevered column systems were not identified as a unique system and were treated by designers as OMF. However, in the 1994 Northridge earthquake, a series of first-story collapses occurred in the Northridge Meadows complex, a series of multi-story residential structures that used cantilevered steel columns adjacent to garage door openings at the first story. Soon after this, the present, more restrictive design provisions for this system were placed into the building code.

SECTION 9
COMPOSITE SYSTEMS

9.1 What is a composite system?

A composite system is one that uses a combination of structural steel and reinforced concrete elements to provide lateral resistance. These systems have the advantage that they can combine the high tensile and flexural strength and rapid erection characteristics of structural steel with the high stiffness and compressive strength of reinforced concrete to obtain economical structures. Design requirements for composite structural systems used to resist seismic forces are contained in Part II of AISC 341, together with AISC 360 and ACI 318, *Building Code Requirements for Reinforced Concrete*. Unlike SLRS of structural steel, which can be designed using either LRFD or ASD procedures, only LRFD procedures can be used for the design of the concrete elements of composite structures, because ACI 318 only contains LRFD-type provisions.

Composite structures can provide very economical solutions and have been used to construct some very tall structures, where the high compressive strength of reinforced concrete columns can be used to maximum advantage. However, many general contractors are reluctant to construct composite structures because of complexities, perceived or real, associated with having multiple trades, including iron workers for structural steel and rod benders and carpenters for reinforced concrete, on site and working on portions of the structure simultaneously. Because of these concerns, composite construction has been used in some regions of the United States and ignored in others.

9.2 What composite SLRS are permitted for seismic design?

There is a wide range of composite steel and concrete SLRS contained in the building code. These include special, intermediate and ordinary moment frames; braced frames; wall systems; and dual systems. A few of the more common types of composite systems include:

- Composite moment frames with steel columns and composite beams. This moment frame system uses steel columns, and steel beams, with composite structural slabs. The structural slab provides a portion of the compression flange for the beam in positive flexure and the reinforcing in the slab provides a portion of the tensile flange behavior in negative flexure. These frames often use seated-type beam connections. Particularly in zones of limited seismicity, by allowing all beam-to-column connections to participate in lateral resistance—using relatively inexpensive connections—they can allow frames to provide required lateral resistance with little increase in the size of members or the cost of the frame. Connections used in these frames are usually considered partially restrained.

- Composite moment frames with reinforced concrete columns and steel beams. This type of moment frame uses conventional reinforced concrete columns, designed in accordance with ACI 318, together with structural steel beams to form moment frames. The connections of the steel beam to the columns must be specially detailed to enable the transfer of stresses between these two elements and to allow development of inelasticity in the desired members, typically the beams. Presently, there are no prequalified composite connections, so designers wishing to use this system must either find prior tests to demonstrate the adequacy of their connection details or perform a program of project-specific qualification testing of their connections. Limited research into the behavior of several types of composite framing connections have been conducted in the past by Leon and Deierlein.

- Composite moment frames and braced frames using concrete-filled hollow structural sections (HSS). In these structures, the concrete fill in the HSS is used to increase the flexural stiffness and axial strength of the tubular steel member.

- Shear-wall systems with reinforced concrete webs and steel boundary elements. In these structures, conventional reinforced concrete walls are used to resist the shears associated with lateral forces, while steel column elements located at the ends of the walls provide the overturning resistance for the wall. This structure type has the advantage of combining the relatively inexpensive construction of reinforced concrete walls with convenient connections for steel platform framing at floor levels, supported by the steel columns.

- Shear-wall systems with composite steel plate and concrete walls. In these systems, a relatively thin steel plate web is adhered to a reinforced concrete wall. The steel plate provides high tensile and shear strength while the concrete provides stiffness and also laterally braces the plate to inhibit buckling.

9.3 What are the advantages of composite systems?

The principal advantage of composite systems is that they combine the best characteristics of structural steel (lightweight, high tensile and flexural strength, and high ductility) with the best characteristics of reinforced concrete (fire resistance and ability to withstand large compressive loads). If properly configured, composite systems can provide extremely economical structural systems with high durability and superior seismic performance characteristics.

9.4 What are the disadvantages of composite systems?

There are a number of disadvantages to the use of composite systems, which have prevented their widespread use in the United States. The first of these is that there has been relatively little research into the seismic behavior of these systems, and as a result, much of the prescriptive design and detailing provisions that exist for structural steel systems are not yet available for composite systems. Also, many engineers and building officials are reluctant to use these systems because they are not familiar with them.

Perhaps more important, many general contractors are reluctant to construct these structures. Construction of a composite structure requires careful coordination of the several trades that perform structural steel construction and reinforced concrete construction. Many contractors and subcontractors will specialize either in structural steel or reinforced concrete construction. They will have the experienced forces to perform one type of construction but not necessarily the other. Thus, multiple contractors must typically be engaged on the site at the same time, and in the same locations, in order to construct a composite structure. Successful construction of such a structure requires the general contractor to carefully coordinate the efforts of these multiple subcontracts. Since few general contractors have actually done this, they do not have a reliable data base of construction costs for such structures and hence, tend to be conservative in estimating probable construction costs. This in turn, discourages design teams from selecting these structural systems.

SECTION 10
IMPORTANT EARTHQUAKES AND BUILDING PERFORMANCE

The seismic design and construction requirements contained in U.S. building codes have been developed over many years, both as a result of laboratory and analytical research and also through observation of the way real structures perform in earthquakes. Following each major earthquake that causes damage to modern engineered construction, engineers and researchers investigate the behavior of typical buildings. When these engineers and researchers observe that certain design and construction practices lead to unacceptable types of damage in buildings, they develop building code provisions to discourage the continued use of these practices in future design and construction. This process, often termed “learning from earthquakes,” has been under way in the United States and worldwide for more than 100 years. This section summarizes some of the more significant lessons that have been learned from past earthquakes with regard to the performance of steel structures and discusses how these observations have been memorialized in building code requirements.

10.1 The 1906 San Francisco earthquake.

The great M7.9 San Francisco earthquake of April 18, 1906, remains one of the worst natural disasters to affect the United States, and had great significance with regard to the development of U.S. building codes. As illustrated in Figure 10-1, at the time of the earthquake, the City of San Francisco was composed of three predominant types of construction: light repetitively framed wood construction; unreinforced brick and stone masonry bearing-wall structures, usually with wood-framed floors; and steel-frame structures with unreinforced masonry infill walls, often with hollow clay tile arch or concrete arch floors. The wood-frame structures were generally less than four stories tall and the masonry bearing-wall structures less than six stories tall. Some of the steel-frame structures were nearly 20 stories tall. Primary lessons learned from this earthquake included the importance of soil type on structural behavior and the benefits of having a complete vertical load-carrying steel frame for seismic resistance.

The importance of site soil effects on earthquake performance of buildings was particularly evident in the behavior of buildings in San Francisco. Much of the urban center of San Francisco was constructed on land that was reclaimed from the surrounding San Francisco Bay. The fill soils used to reclaim this land were of mixed characteristics, consisting of debris from building construction; excavation spoils from building basements on the adjacent dry land; and even the rotting hulls of ships, abandoned in the bay as the crews headed for the gold fields following the 1849 discovery. In

Fig. 10-1. San Francisco prior to the great earthquake of April 1906. (Photo courtesy of Library of Congress)

the period immediately after the earthquake, and before the ensuing fires destroyed much of the city, observers noted that buildings constructed on this "made" or "infirm" ground performed far worse than buildings that had been constructed on the natural ground that defined San Francisco prior to the gold rush. This observation was included in early building code provisions for seismic resistance, developed in the 1920s, which required higher design forces for buildings sited on infirm sites. As time progressed, memory of these effects faded, and the building codes of the 1940s no longer included this factor. It was to be rediscovered in the 1970s and 1980s and re-instituted into present building codes.

One of the most startling observations arising from the San Francisco earthquake was related to the performance of different types of building construction. As seen in Figure 10-2, following the earthquake and ensuing fire, the only buildings in the commercial center of San Francisco that remained standing were those constructed with complete vertical load-carrying steel frames and infill masonry walls. This observation led to the eventual requirement contained in building codes that tall buildings have complete vertical load-carrying space frames and that buildings taller than 240 ft have moment-resisting space frames as part of their seismic load resisting system.

10.2 The 1933 Long Beach earthquake.

On March 10, 1933, a moderate-magnitude M6.3 earthquake struck near Long Beach, California, causing extensive damage to unreinforced masonry buildings in the city and killing 115 people. Many school buildings were among the damaged structures; one such school is shown in Figure 10-3. The massive damage to unreinforced masonry buildings and schools prompted California to adopt legislation prohibiting the further construction of unreinforced masonry buildings in the state and also empowering the Office of the State Architect to adopt occupancy-specific design, construction and quality assurance requirements for the construction of schools. This legislation marked the beginning of two important trends still present in current building codes. The first of these was the prohibition against the use of certain types of structural systems, in this case, unreinforced masonry. The second was the recognition that some buildings are more important than others and, therefore, should be designed and constructed with greater precautions to protect the safety of the public.

10.3 The 1940 Imperial Valley and 1952 Kern County earthquakes.

The 1940 Imperial Valley earthquake, with a magnitude of nearly M7.0, occurred in the California desert, east of San Diego. This earthquake affected few buildings in this sparsely populated region. However, the USGS did obtain a high-quality, three-axis, strong ground motion recording from this event. For many years thereafter, the Imperial Valley strong motions records were among the few available to researchers, and much of the work performed by early earthquake engineering researchers in developing response spectra contained in building codes was based on these recorded motions.

The 1952 Kern County earthquake was a large event, with a magnitude of nearly M7.4. Located in the arid area east of Bakersfield, California, this earthquake also caused little

Fig. 10-2. Downtown San Francisco following the earthquake and fires of 1906. (Photo courtesy of Library of Congress)

building damage of significance, but it did result in extensive damage to oil field and refining facilities in the region. Following this earthquake, ASCE and SEAOC formed a joint committee, known as the Separate 66 Committee, to make recommendations for seismic design provisions in building codes. The recommendations of the Separate 66 Committee eventually resulted in adoption by the building codes of requirements to determine seismic design forces for building based on spectral response analysis concepts.

10.4 The 1971 San Fernando earthquake.

The M6.6 San Fernando earthquake of February 9, 1971, though not of great magnitude, was one of the most significant events with regard to its effect on building codes. Prior to the 1971 earthquake, the seismic provisions in building codes were largely limited to specification of minimum design lateral forces and contained few requirements related to structural detailing. Because a large-magnitude earthquake had not affected California in nearly 20 years prior to this event, engineers felt confident that the building codes in effect at the time were capable of providing reliable protection of buildings in earthquakes. However, the San Fernando earthquake severely damaged many modern, code-conforming buildings. Among the most famous of these damaged buildings was the Olive View Hospital, a large multi-building complex located near the epicenter of this earthquake (Figure 10-4). This complex consisted of a series of reinforced concrete frame buildings. Although the 1967 edition of the UBC contained provisions for ductile reinforced concrete frame design, these requirements were not mandatory and had not been included in the hospital's design. One of the buildings, which housed ambulances and emergency vehicles, collapsed. Stair towers separated and fell away from the main hospital building, and a single-story mechanism formed in the columns at the first level of the main structure, resulting in large permanent drift and leaving the building unrepairable.

In response to this damage, and damage sustained by other hospitals and commercial buildings, a number of major revisions were introduced into the building codes in the following years. Perhaps the most important of these changes was the recognition of the importance of ductile detailing to seismic performance. The voluntary provisions for ductile concrete moment frames were made mandatory in regions of high seismicity, and similar provisions began to be developed and introduced into the code for other structural systems, essentially resulting in the precedent for the Special, Intermediate and Ordinary classifications of seismic load resisting systems contained in building codes today.

Another important change related to more formal consideration of building occupancy when determining the seismic design requirements. Following this earthquake, the concept of occupancy categories was introduced into the building code, with higher design forces required for the design of hospitals and other buildings deemed to be essential to the public safety.

Finally, as with past earthquakes, following the San Fernando event, engineers once again recognized that the types of soil present at a site had great significance with regard to the intensity and character of ground motions experienced by buildings. This resulted in the formal introduction of soil profile types, or as called in today's codes, Site Classes, into the determination of seismic design forces and other requirements for buildings.

Fig. 10-3. Compton Junior High School, Long Beach, California, 1933. (Photo courtesy of U.S. Geologic Survey)

10.5 The 1979 Imperial Valley earthquake.

The M6.4 Imperial Valley earthquake of October 15, 1979, affected relatively few buildings due to the sparse population of the affected region, near the California–Mexico border. However, one building, the six-story Imperial County Services Building (Figure 10-5) did experience noteworthy damage. This six-story concrete shear wall building had an out-of-plane offset between the shear walls and frames above the first story and those below, in order to accommodate an arcade feature at the ground level. Overturning forces from the shear walls above the first story crushed the first story columns immediately below the walls and frames (Figure 10-6).

Research into the behavior of this building led to the present code requirement to design columns beneath discontinuous walls and frames for the amplified forces that consider the overstrength of the structure above. Later, the building code applied this same requirement to other irregularities.

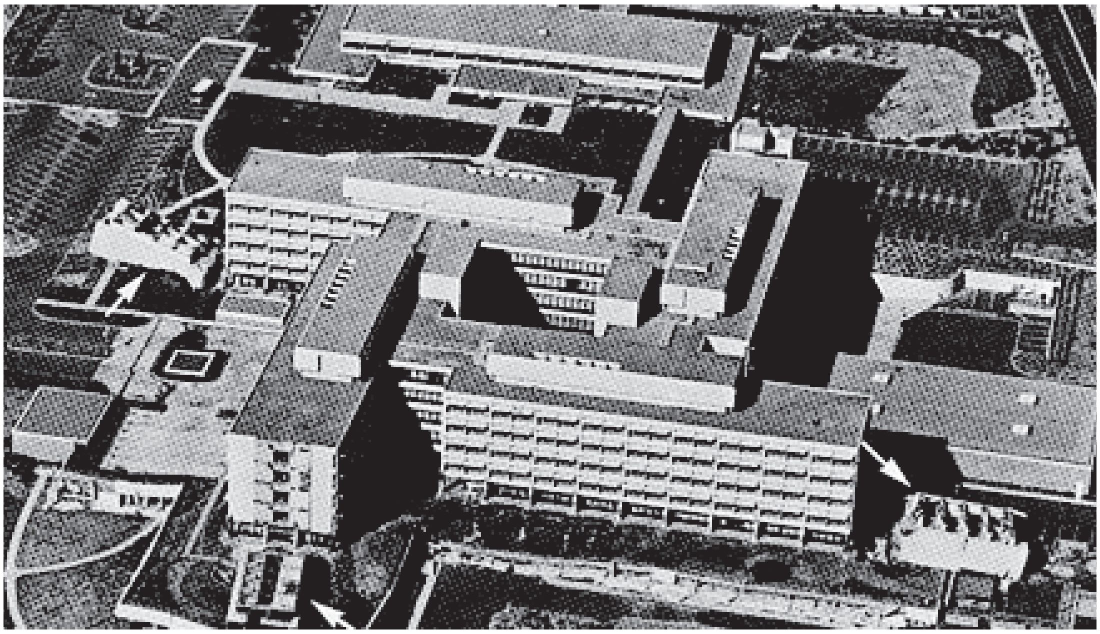


Fig. 10-4. Partial collapse of the Olive View Hospital, 1971 San Fernando earthquake. (Photo courtesy of U.S. Geologic Survey)

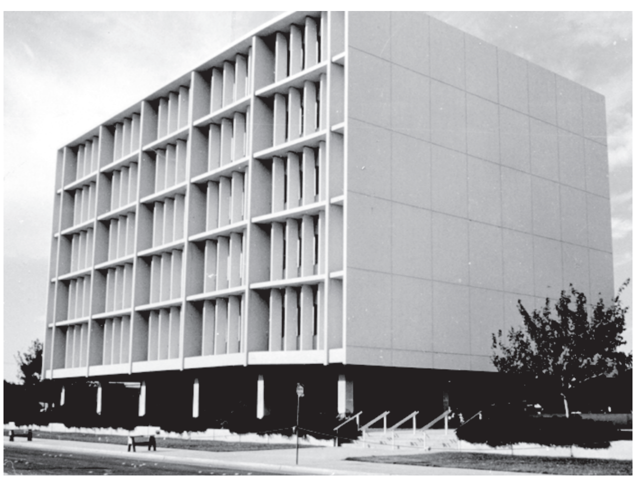


Fig. 10-5. Imperial County Services Building, El Centro, California. (Photo courtesy of U.S. Geologic Survey)

Fig. 10-6. Crushed columns at base of Imperial County Services Building. (Photo courtesy of U.S. Geologic Survey)

10.6 The 1985 Mexico City earthquake.

Prior to the great M8.1 Mexico City earthquake of September 19, 1985, there had been little record of any significant damage to steel frame buildings. However, in this earthquake, two high-rise steel frame buildings at the Piño Suarez complex, one 22 stories tall and the other 16 stories tall, collapsed. These buildings were braced steel frame structures that utilized built-up box section columns. Investigation of damage sustained by the structures in the complex that remained standing suggested that overturning forces imposed on the columns resulted in local buckling of plate sections in the built-up box section columns, which then led to failure of the seam welds in the boxes (Figure 10-7). Once these seams opened up, the columns buckled, resulting in the collapse of two of the four structures in the complex.

This observation led to the design requirement to proportion the columns in steel seismic load resisting systems with adequate strength to resist the maximum axial forces that can be delivered to the columns, considering the overstrength of the structural system supported above, whether or not the structural system is irregular.

10.7 The 1987 Whittier Narrows earthquake.

The M5.9 Whittier Narrows earthquake of October 17, 1987, was a relatively modest event, both in size and effect. However, it did cause damage to some modern buildings in a region where the engineering community was actively engaged in earthquake observation, prompting several code changes. One of the most significant of these changes was based on observation of damage sustained by the California Federal Savings Company's data processing center. This steel braced-frame building employed chevron-pattern braces. As commonly happens with such frames, the braces buckled in compression (Figure 10-8) and the floor beams at the apex of the chevrons were bent downward, causing damage to the floor systems. Observation of this damage led to the introduction of provisions requiring design of beams at the apex of chevron-pattern braces for the unbalanced forces that result following buckling of one of these braces.

10.8 The 1989 Loma Prieta earthquake.

The 1989 M7.1 Loma Prieta earthquake, occuring approximately 70 miles south of San Francisco, caused relatively little damage to modern structures designed to recent editions of the building code, though it caused extensive damage to older buildings and structures. Although few modern buildings were seriously damaged by this event, there was a large array of strong motion recording instrumentation in the San Francisco Bay area, providing a wealth of data on the character of ground shaking at sites located at different distances and azimuths from the zone of fault rupture, as

Fig. 10-7. Failed built-up box column in one of the surviving Piño Suarez Towers. (Photo courtesy of John Osteraas)

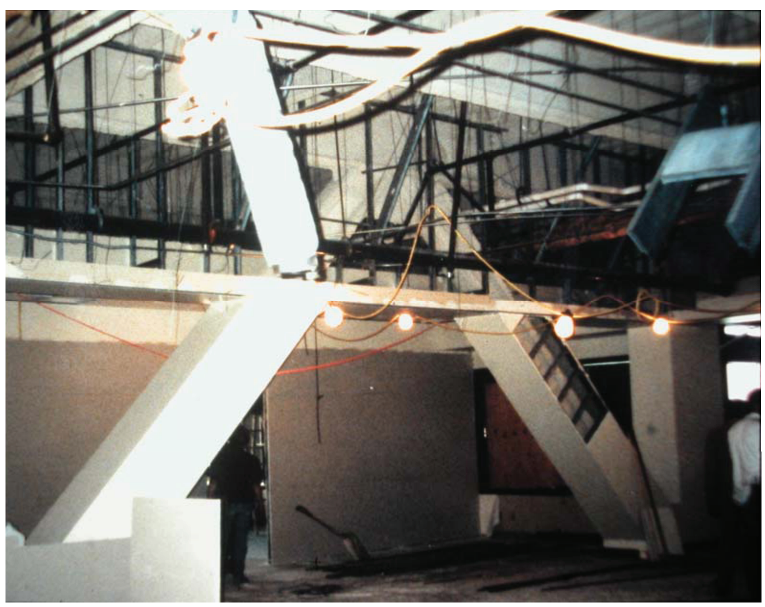


Fig. 10-8. Buckled chevron-pattern brace at the California Federal Savings data processing center. (Photo courtesy of P. Yanev)

well as having differing site conditions. This data, combined with data available from earlier events, made it possible for geotechnical engineers and seismologists to develop the Site Class factors and associated spectral shape modifications contained in present-day building codes. In addition, the wealth of ground shaking data obtained from this earthquake made it possible for seismologists to develop and calibrate numerical models that could be used to simulate the shaking likely to be experienced at a site from a specific scenario earthquake.

The ground motion data obtained in this earthquake was also key to the determination that the character of ground shaking in the region within the near field—that is, within a few kilometers of the zone of fault rupture—is not only stronger, but also significantly different from the character of shaking experienced farther from the rupture zone. Recordings obtained in this event, together with the limited near-fault recordings available from other earthquakes (such as the 1971 San Fernando and 1992 Landers and Big Bear events), allowed seismologists to identify the pulse-like characteristics of near-field motions, as well as the dependence of these impulses on the direction of fault rupture, relative to a site, and the orientation of the instrument. However, it was not until after the 1994 Northridge earthquake, when these effects were again noted, that the building code was actually modified to account for these effects.

10.9 The 1994 Northridge earthquake.

The M6.7 earthquake that struck Northridge, California, on January 17, 1994, was one of the most significant earthquakes of the past century with regard to the wealth of engineering data that was obtained and analyzed and subsequently implemented into the building codes. This is because, like the San Francisco Bay area in 1989, the affected region had many strong ground motion instruments present, but also, unlike the Loma Prieta earthquake, this event damaged many modern code-conforming buildings.

The Northridge earthquake provided valuable earthquake experience data on the performance of four types of structures: concrete tilt-up buildings with wood roofs, precast concrete parking structures, braced steel frames and moment-resisting steel frames. As a result of the observations of damage that occurred in this earthquake, extensive revisions were made to the 1997 editions of the UBC, as well as the NEHRP Provisions, which form the basis for seismic design requirements contained in SEI/ASCE 7. The changes to the two code documents were essentially identical.

With regard to braced steel frames, the observation of fractures in HSS braces, following buckling, resulted in significant restrictions on the permissible width-thickness ratios for brace elements. It also resulted in severe restrictions on the use of OCBF.

Perhaps the most significant lessons learned were associated with the unanticipated discovery of fractures in the welded joints of modern steel moment frames (see Figures 6-2, 6-3, and 6-4). These fractures were attributed to a variety of factors, including connection geometries that resulted in stress concentrations and high restraint, limiting ductility; wide variations in the yield and tensile strengths of the common ASTM A36 and A572 grades of material used in building construction at that time; the low toughness of weld filler metals commonly used in steel construction; and poor adherence to the requirements of the AWS welding code when making welded joints. Another important contributing factor to these unexpected failures was that the connection practices commonly in use prior to the earthquake had been validated years earlier by testing of specimens that were much lighter than those present in the damaged buildings. Over the years since the initial research, design practice had evolved away from the use of highly redundant frames with relatively small members to framing systems with relatively few participating members and use of very large sections. The earlier testing was not applicable to these heavier frames, but this was not understood until after the earthquake and discovery of the damage.

These observations resulted in a wealth of changes in the building codes and design practice as well as a major rewrite of AISC 341 and the introduction of the AWS D1.8 seismic supplement to AWS D1.1. Significant changes included the introduction of the new ASTM A992 grade of structural steel, with controlled yield strengths and yield to tensile ratios; requirements to demonstrate (through laboratory testing of full-scale specimens) that moment connections are capable of attaining minimum inelastic deformation demands; requirements to use weld filler metals with minimum rated notch toughness in seismic load resisting systems; and requirements to remove weld backing bars and weld tabs from critical joints.

Another major feature introduced into the building codes following the Northridge earthquake was the requirement to quantify the redundancy inherent in a structural design and to adjust the design seismic forces and permissible drifts for the structure, based on this redundancy. This was based on the observation that many modern structures that had been severely damaged in the earthquake were less redundant than earlier structures that performed better.

SECTION 11
FUTURE TRENDS AND RESEARCH

Throughout the 20th century, the primary intent of seismic design provisions in building codes was to avoid earthquake-induced damage in buildings that would pose a significant risk to safety, while permitting the most economical designs that could accomplish this goal. Thus, building code provisions were developed that would permit some types of damage to occur, but protect against damage likely to lead to either local or partial collapse or the generation of dangerous falling debris. When these building codes were first developed, the technical community did not have a good understanding of the character of ground shaking produced by earthquakes, its magnitude, the dynamic response characteristics of structures, or concepts of nonlinear behavior. Many of the building code provisions were based on observation of damage that could be attributed to specific design and construction details rather than real understanding of the cause of the damage. As time passed, and engineering knowledge improved, the observations of damage and resulting requirements introduced into the building codes improved, with more technical basis for the requirements in the code. Today's codes still seek primarily to protect life safety, rather than minimize damage, and seek to do so through a variety of prescriptive criteria based on observation, as well as laboratory and analytical research.

Prior to the 1971 San Fernando earthquake, building code provisions were primarily developed on an ad hoc and voluntary basis by industry associations and volunteer engineers. Following the San Fernando earthquake, the federal government has taken an increasingly prominent role in supporting the development and adoption of reliable building codes, with the goal of minimizing future losses from earthquakes. This has included significant financial support of earthquake engineering and seismologic research, as well as direct support of projects like the SAC Joint Venture that develop recommended building code provisions. For the past 10 years, the annual federal earthquake budget, authorized under the National Earthquake Hazards Reduction Program, has been on the order of $70 million, administered through the National Institute of Building Sciences (NIBS), the Federal Emergency Management Agency (FEMA), the USGS and the National Science Foundation (NSF).

NSF funds primary research at the university level. Over the past 30 years, a significant portion of this funding has been focused on the maintenance of several national earthquake engineering research centers, including the Earthquake Engineering Research Center at the University of California at Berkeley (1975–1985); the National Center for Earthquake Engineering Research at the State University of New York (1986–1996); and in the period 1996–2006, three centers, including the Pacific Earthquake Engineering Research Center, Mid-America Engineering Research Center and Multidisciplinary Earthquake Engineering Research Center. This research has spawned numerous innovations that have become common in earthquake engineering practice, including ductile detailing of concrete structures, improved connections for moment frames, base isolation technology, energy dissipation technology, and the many computing tools that now are regularly used in design practice. NSF continues to fund earthquake engineering research through the National Earthquake Engineering System (NEES), a series of research facilities across the United States that are linked by data communication networks that enable researchers at one facility to collaborate and perform investigations at another.

The USGS has funded a wealth of programs focused on characterizing the seismic hazard, that is, the risk of incurring strong ground motion, throughout the United States. Much of this research takes the form of installing and maintaining strong ground motion instruments and analyzing the recorded motions when earthquakes occur. The USGS efforts have produced the national seismic hazard maps contained in the building code, which are updated every three to five years. Important to structural designers, USGS maintains an Internet-based application that allows determination of design spectral response accelerations in accordance with the building code, based on geographic coordinates and information on the Site Class. USGS has also developed a number of tools, such as SHAKEMAP, that allow rapid assessment of the intensity of ground motion throughout a region, when an earthquake occurs. This is useful to planners, emergency responders and disaster recovery agencies.

NIST, as a subagency within the Department of Commerce, is charged with facilitating the competitiveness of the American economy. In this role, NIST provides cooperative funding with private interests to develop technologies that can be commercialized. In this role, NIST has partnered with AISC to perform research that supported the development of seismic design guidelines and worked with the Pankow Company to support the development of precast hybrid moment frames, and has participated in other, similar ventures.

FEMA's primary role in earthquake research is the development of tools that will promote mitigation of earthquake risk. FEMA has principally fulfilled this role through funding of applied research projects that resulted in the development of design guidelines and tools. In this role, FEMA sponsored the development and maintenance of the NEHRP Provisions, which form the basis of the seismic design provisions contained in SEI/ASCE 7. FEMA funding has also been used to assist AISC and other similar industry groups to develop design criteria for new structural systems.

Present research activities are focused on three basic areas: performance-based design, development of damage-resistant systems, and improvement in our ability to better predict the occurrence and intensity of future earthquakes. This latter activity is principally the responsibility of USGS, while the other agencies are concentrating in the other two areas. The concept of damage-resistant structural systems is a new one and is counter to the philosophy inherent in the building codes of designing to avoid structural collapse and the attendant loss of life. Several things are promoting the research into damage-resistant structural systems. The first of these is a desire on the part of the federal government to avoid future disasters associated with earthquakes, which they feel can best be done by encouraging society to build more damage-resistant construction. Another important factor in this regard is that large portions of the U.S. population and economy are located in areas that are subject to severe earthquakes. Finally, our understanding of the way structures respond to earthquakes and our ability to predict this behavior make it possible, for the first time, to design such structures. Examples of damage-tolerant systems that have been developed in recent years include seismic isolation systems, energy dissipation systems and self-centering frames and walls.

Over the past 15 years, performance-based design has received increasing attention in the research community. The concept of performance-based design is that a designer can be inventive in the combinations of structural framing systems and detailing chosen for a structure, rather than performing designs by adhering to prescriptive criteria contained in the building codes. However, this approach presumes that the designer is capable of demonstrating, typically through simulation, that the structure is capable of performing acceptably. As our understanding of the likely character and intensity of future ground shaking—and our ability to use advanced computing techniques to simulate structural performance in earthquakes—improves, the ability to actually implement performance-based design is becoming more practical. As this trend continues, designers will find that they are no longer constrained to use only certain types of structural systems and configurations, or to adhere to minimum design base shears, drift or detailing criteria, thus providing more freedom in the design of structures of the future.

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