EXISTING BUILDINGS–EVALUATION AND RETROFIT 8

by William Holmes

8.1 INTRODUCTION

It is widely recognized that the most significant seismic risk in this country resides in our existing older building stock. Much of the country has enforced seismic design for new buildings only recently; even on the West Coast, seismic codes enforced in the 1960s and even into the 1970s are now considered suspect. Although there are sometimes difficulties in coordinating seismic design requirements with other demands in new construction, the economical, social, and technical issues related to evaluation and retrofit of existing buildings are far more complex.

8.1.1 Contents of Chapter

This chapter describes the many issues associated with the risk from existing buildings, including common building code provisions covering older buildings, evaluation of the risks from any one given building and what levels of risk are deemed acceptable, and methods of mitigation of these risks through retrofit. A FEMA program to provide methods to mitigate the risk from existing buildings has been significant in advancing the state of the art, and this program is described in some detail, particularly the model building types used in most, if not all, of the FEMA documents.

8.1.2 Reference to Other Relevant Chapters

The basic concepts used for seismic design or estimation of seismic performance are the same for any building. Thus, the principles described in Chapters 4, 5, and 7 are applicable for older, potentially hazardous buildings. The development of seismic systems as seen through examples of buildings in the San Francisco Bay Area is particularly relevant to the issues covered in this chapter, because systems typically evolved due to poor performance of predecessors.

Nonstructural systems in buildings create the majority of dollar loss from buildings in earthquakes, although the quality of structural performance affects the level of that damage. Seismic protection of nonstructural systems, both for design of new buildings and for consideration in older buildings, is covered in Chapter 9.

8.2 BACKGROUND

In every older building, a host of "deficiencies" is identified as the state of the art of building design and building codes advances. Code requirements change because the risk or the expected performance resulting from the existing provisions is deemed unacceptable. Deficiencies are commonly identified due to increased understanding of fire and life safety, disabled access, hazardous materials, and design for natural hazards. Thus, it is not surprising that many of the older buildings in this country are seismically deficient, and many present the risk of life-threatening damage. It is not economically feasible to seismically retrofit every building built to codes with no or inadequate seismic provisions, nor is it culturally acceptable to replace them all. These realities create a significant dilemma: How are the buildings that present a significant risk to life safety identified? How is the expected performance predicted for older buildings of high importance to businesses or for those needed in emergency response? How can we efficiently retrofit those buildings identified as high risk?

The term **seismic deficiency** is used in this chapter as a building characteristic that will lead to unacceptable seismic damage. Almost all buildings, even those designed to the latest seismic codes, will suffer earthquake damage given strong enough shaking; however, damage normally considered acceptable should not be expected in small events with frequent occurrence in a given region, and should not be life threatening. Damage may be judged unacceptable due to resulting high economic cost to the owner or due to resulting casualties. Therefore, conditions that create seismic deficiencies can vary from owner to owner, from building to building, and for different zones of seismicity. For example, unbraced, unreinforced brick masonry residential chimneys are extremely vulnerable to earthquake shaking and should be considered a deficiency anywhere that shaking is postulated. On the other hand, unreinforced brick masonry walls, infilled between steel frame structural members, are expected to be damaged only in moderate to strong shaking and may not be considered a deficiency in lower seismic zones. Seismic deficiencies identified in this chapter generally will cause premature or unexpected damage, often leading to threats to life safety, in moderate to strong shaking. Buildings in regions of lower seismicity that expect Modified Mercalli Intensity (MMI) levels of not more than VII, or peak ground accelerations (PGA) of less than 0.10g (g = acceleration of gravity), may need special consideration.

Of course, every building with completed construction is "existing." However, the term **existing building** has been taken to mean those buildings in the inventory that are not of current seismic design. These groups of buildings, some of which may not be very old, include buildings with a range of probable performance from collapse to minimal damage. In this chapter, the term "existing building" is used in this context.

8.2.1 Changes in Building Practice and Seismic Design Requirements Resulting in Buildings that are Currently Considered Seismically Inadequate

Chapter 7 documents in detail how building systems have evolved in the San Francisco Bay Area. This evolution was probably driven more by fire, economic, and construction issues than by a concern for seismic performance, at least in the first several decades of the twentieth century, but many changes took place. Similarly, Chapter 6 gives a brief history of the development of seismic codes in the United States. It is clear that for many reasons, building construction and structural systems change over time. In the time frame of the twentieth century, due to the rapid increase in understanding of the seismic response of buildings and parallel changes in code requirements, it should be expected that many older buildings will now be considered seismically deficient.

Seismic codes in this country did not develop at all until the 1920s, and at that time they were used voluntarily. A mandatory code was not enforced in California until 1933. Unreinforced masonry (URM) buildings, for example, a popular building type early in the twentieth century and now recognized as perhaps the worst seismic performer as a class, were not outlawed in the zones of high seismicity until the 1933 code, and continued to be built in much of the country with no significant seismic design provisions until quite recently. Figure 8-1 shows an example of typical URM damage. The first modern seismic codes were not consistently applied until the 1950s and 1960s, and then only in the known regions of high seismicity. Of course, not all buildings built before seismic codes are hazardous, but most are expected to suffer far more damage than currently built buildings

Even buildings designed to "modern" seismic codes may be susceptible to high damage levels and even collapse. Our understanding of seismic response has grown immensely since the early codes, and many building

Figure 8-1: Example of buildings with no code design. Damage shows classic URM deficiencies.

characteristics that lead to poor performance were allowed over the years. For example, concrete buildings of all types were economical and popular on the West Coast in the 1950s and 1960s. Unfortunately, seismic provisions for these buildings were inadequate at the time, and many of these buildings require retrofit. Highlights of inadequacies in past building codes that have, in many cases, created poor buildings are given below.

- ### Changes In Expected Shaking Intensity and Changes in Zoning

  Similar to advancements in structural analysis and the understanding of building performance, enormous advancements have been made in the understanding of ground motion, particularly since the 1950s and 1960s. The seismicity (that is, the probability of the occurrence of various-sized earthquakes from each source) of the country, the likely shaking intensity from those events depending on the distance from the source and the local soil conditions, and the exact dynamic nature of the shaking (the pattern of accelerations, velocities, or displacements) are all far better understood. These advancements have caused increases in seismic design forces from a factor of 1.5 in regions very near active faults (on the West Coast) to a factor of 2 to 3 in a few other areas of the country (e.g. Utah; Memphis, Tennessee). The damage to the first Olive View Hospital (Figure 8-2), in addition to other issues, was a result of inadequate zoning.

- ### Changes in Required Strength or Ductility

  As discussed in Chapter 4, the required lateral strength of a seismic system is generally traded off with the **ductility** (the ability to deform inelastically—normally controlled by the type of detailing of the components and connections) of the system. Higher strength requires lower ductility and vice versa. The most significant changes in codes—reflecting better understanding of minimum requirements for life safety—are general increases in both strength and ductility. Many building types designed under previous seismic provisions, particularly in the 1950s, 1960s, and 1970s, are now considered deficient, including most concrete-moment frames and certain concrete shear walls, steel-braced frames, and concrete tilt-ups.

Other buildings designed with systems assumed to possess certain ductility have been proven inadequate. Figure 8-3 shows typical steel moment-frame damage in the Northridge earthquake caused by brittle behavior in a structural system previously thought to be of high ductility.

- Recognition of the Importance of Nonlinear Response

Historically, a limited amount of damage that absorbed energy and softened the building, thus attracting less force, was thought o reduce seismic response. Although this is still true, it is now recognized that the extent and pattern of damage must be controlled. Early codes required the design of buildings for forces three to six times less than the **elastic demand** (the forces that the building would see if there was no damage), assuming that the beneficial characteristics of damage would make up the difference. Unfortunately, buildings are not uniformly damaged, and the change in structural properties after damage (nonlinear response) often will concentrate seismic displacement in one location. For example, if the lower story of a building is much more flexible or weaker than the stories above, damage will concentrate at this level and act as a fuse, never allowing significant energy absorption from damage to the structure above. This concentrated damage can easily compromise the gravity load-carrying capacity of the structure at that level, causing collapse. Similarly, concrete shear walls were often "discontinued" at lower floors and supported on columns or beams. Although the supporting structure was adequately designed for code forces, the wall above is often much stronger than that and remains undamaged, causing concentrated and unacceptable damage in the supporting structure.

A final example of this issue can be seen by considering torsion. As explained in Chapter 4, Section 4.11, torsion in a building is a twisting in plane caused by an imbalance in the location of the mass and resisting elements. Older buildings were often designed with a concentration of lateral strength and stiffness on one end—an elevator/stair tower, for example—and a small wall or frame at the other end to prevent torsion. However, when the small element is initially damaged, its strength and stiffness changes, and the building as a whole may respond with severe torsion.

Current codes contain many rules to minimize configurations that could cause dangerous nonlinear response, as well as special design rules for elements potentially affected (e.g., columns supporting discontinuous shear walls). Olive View Hospital featured a weak first story in the main building, causing a permanent offset of more than one foot and near collapse; discontinuous shear walls in the towers caused a failure in the supporting beam and column frame, resulting in complete overturning of three of the four towers. Figure 8-4 shows a typical "tuck-under" apartment building in which the parking creates a weak story.

8.2.2 Philosophy Developed for Treatment of Existing Buildings

Building codes have long contained provisions to update life-safety features of buildings if the occupancy is significantly increased in number or level of hazard (transformation of a warehouse to office space, for example). As early as the mid-1960s, this concept started to be applied to seismic systems. Many older buildings contained entire structural systems no longer permitted in the code (e.g., URM, poorly reinforced concrete walls), and it quickly became obvious that 1) these components could not be removed, and 2) it was impractical and uneconomical to replace all older buildings. The "new" code could therefore not be applied directly to older buildings, and special criteria were needed to enable adaptive reuse while meeting the need to protect life safety of the occupants. In some cases, an entirely new and code-complying lateral system was installed, while leaving existing, now prohibited, construction in place. (This procedure was used in many school buildings in California after the Field Act was passed in 1933—up until the school seismic safety program was essentially completed in the 1960s.) This procedure proved very costly and disruptive to the building and was thought to discourage both improved seismic safety and general redevelopment.

Figure 8-2: Olive View Hospital. A brand new facility that was damaged beyond repair in the 1971 San Fernando earthquake due to a shaking intensity that exceeded what was expected at the site and a design that, although technically complying with code at the time, contained several structural characteristics now considered major deficiencies. The lateral system contained nonductile concrete frames, discontinuous shear walls, and a significant weak story.

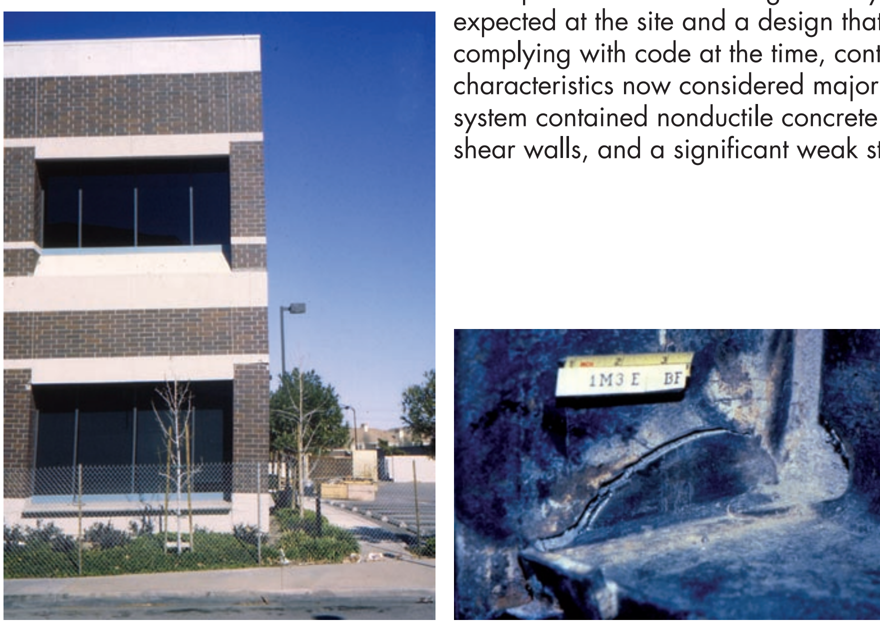


Figure 8-3: An example of a recently designed building with seismic deficiencies not understood at the time of design. In this case, the deficiency was "pre-Northridge" moment-frame connections, which proved to be extremely brittle and unsatisfactory. Hundreds, if not thousands, of these buildings were designed and built in the two decades before the Northridge earthquake (1994).

Figure 8-4: A 1970s building type with a deficiency not prohibited by the code at the time—a tuck-under apartment with parking at the ground level, creating a weak story. Many of these buildings collapsed in the Northridge earthquake. Sixteen deaths occurred in the Northridge Meadows apartments, or 42% of those directly killed by the earthquake.

Figure 8-5: A tuck-under similar to Figure 8-4, but much more modern and designed at a time when the weakness of the parking level was more understood. In this case, the detailing of the small wood shear walls at that level was poor. This practice created another set of deficient "existing" buildings.

Figure 8-6: A tilt-up damaged in 1994. Despite suspicions that the code-required roof-to-wall ties were inadequate, it took several code cycles and incremental increases in requirements to obtain adequate code provision. Thousands of tilt-ups with inadequate connections exist in the West, although several jurisdictions are actively requiring retrofits.

SOURCE: LLOYD CLUFF

A philosophy quickly developed suggesting that existing buildings be treated differently from new buildings with regard to seismic requirements. First, archaic systems and materials would have to be recognized and incorporated into the expected seismic response, and secondly, due to cost and disruption, seismic design force levels could be smaller. The smaller force levels were rationalized as providing minimum life safety, but not the damage control of new buildings, a technically controversial and unproven concept, but popular. Commonly existing buildings were then designed to 75% of the values of new buildings—a factor that can still be found, either overtly or hidden, in many current codes and standards for existing buildings.

Occasionally, early standards for existing buildings incorporated a double standard, accepting a building that passed an evaluation using 75% of the code, but requiring retrofits to meet a higher standard, often 90% of the code.

8.2.3 Code Requirements Covering Existing Buildings

As the conceptual framework of evaluation and retrofit developed, legal and code requirements were also created. These policies and regulations can be described in three categories; active, passive, and post-earthquake. Active policies require that a defined set of buildings meet given seismic criteria in a certain time frame—without any triggering action by the owner. For example, all bearing-wall masonry buildings in the community must meet the local seismic safety criteria within ten years. Passive policies require minimum seismic standards in existing buildings only when the owner "triggers" compliance by some action—usually extensive remodeling, reconstruction, or addition. Post-earthquake policies developed by necessity after several damaging earthquakes, when it became obvious that repairing an obviously seismically poor building to its pre-earthquake condition was a waste of money. It then became necessary to develop triggers to determine when a building could simply be repaired and when it had to be repaired and retrofitted as well.

● Passive Code Provisions

As noted above, the development of requirements to seismically update a building under certain conditions mimicked economic and social policies well-established in building codes. Namely, the concept crystallized that if sufficient resources were spent to renew a building, particularly with a new occupancy, then the building should also be renewed seismically. Seismic "renewal" was defined as providing life safety, but not necessarily reaching the performance expected from a new building. A second kind of trigger—that could be termed "trigger of opportunity"—has also been used in some communities. These policies try to take advantage of certain conditions that make seismic improvements more palatable to an owner, such as retrofit of single-family dwellings at point of sale or requiring roof diaphragm upgrades at the time of re-roofing.

Triggers based on alterations to the building are by far the most common and will be discussed further here. These policies are somewhat logical and consistent with code practice, but they created two difficult socio-economic-technical issues that have never been universally resolved. The first is the definition of what level of building renewal or increase in occupancy-risk triggers seismic upgrading. The second is to establish the acceptable level of seismic upgrading.

Most typically, the triggering mechanisms for seismic upgrade are undefined in the code and left up to the local building official. The *Uniform Building Code*, the predecessor to the IBC in the western states, waffled on this issue for decades, alternately inserting various hard triggers (e.g., 50% of building value spent in remodels) and ambiguous wording that gave the local building official ultimate power. The use of this mechanism, whether well defined in local regulation or placed in the hands of the building official, ultimately reflects the local attitude concerning seismic safety. Aggressive communities develop easily and commonly triggered criteria, and passive or unaware communities require seismic upgrade only in cases of complete reconstruction or have poorly defined, easily negotiated triggers. For more specific information on seismic triggers in codes, see the accompanying sidebar.

When seismic improvement is triggered, the most common minimum requirement is life safety consistent with the overall code intent. However, the use of performance-based design concepts to establish equivalent technical criteria is a recent development and is not yet universally accepted. As indicated in the last section, the initial response to establishing minimum seismic criteria was to use the framework of the code provisions for new buildings with economic and technical adjustments as required. These adjustments included a lower lateral force level (a pragmatic response to the difficulties of retrofit), and special consideration for materials and systems not allowed by the provisions for new buildings.

Box 1 Seismic Triggers in Codes

Events or actions that require owners to seismically retrofit their buildings are commonly called **triggers**. For example, in many communities, if an owner increases the occupancy risk (as measured by number of occupants, or by use of the building), they must perform many life-safety upgrades, including seismic ones. However, for practical and economic reasons, seldom does this trigger require conformance with seismic provision for new buildings, but rather with a special **life-safety** level of seismic protection, lower than that used for new buildings.

The code with the longest history in high-seismic regions, the *Uniform Building Code* (UBC), has long waffled on this issue. Besides the traditional code life-safety trigger based on clear-cut changes in occupancy, this code over the years has included provisions using hard triggers based on the cost of construction, and soft language that almost completely left the decision to the local building official. The last edition of this code, the 1997 UBC, basically allowed any (non-occupancy related) alteration as long as the seismic capacity was not made worse.

The codes and standards that will replace the UBC are based on a federally funded effort and published by FEMA as the NEHRP *Provisions*. These codes include the *International Building Code* (IBC), the National Fire Protection Agency (NFPA) and ASCE 7, a standard covering seismic design now ready for adoption by the other codes. This family of regulations has a common limit of a 5% reduction in seismic capacity before "full compliance" is required. This reduction could be caused by an increase in mass (as with an addition) or a decrease in strength (as with an alteration that places an opening in a shear wall). Full compliance in this case is defined as compliance with the provisions for new buildings that do not translate well to older buildings. It is unclear how this will be interpreted on the local level.

Many local jurisdictions, however, have adopted far more definitive triggers for seismic retrofit. San Francisco is a well-known example, perhaps because the triggers are fairly elaborately defined and because they have been in place for many years. In addition to the traditional occupancy-change trigger, San Francisco requires conformance with seismic

(continued over)

Seismic Triggers in Codes (continued)

provisions specially defined for existing buildings when substantial nonstructural alterations are done on 2/3 or more of the number of stories within a two-year period, or when substantial structural alterations, cumulative since 1973, affect more than 30% of the floor and roof area or structure.

The City of Portland, Oregon requires a seismic upgrade of URMs when the cost of construction exceeds $30/sf for a one story building, or $40/sf for buildings greater than two stories.

Although most jurisdictions leave this provision purposely loose, some also have adopted definitive triggers based on cost of construction, on the particular building type, and on various definitions of significant structural change.

Government, in some cases, has been much more aggressive in setting triggers to activate seismic retrofit, perhaps to create a lawful need for funds which otherwise would be difficult to obtain. The state of California has set a definitive list of seismic triggers for state-owned buildings: a) alteration cost exceeding 25% of replacement cost; b) change in occupancy; c) reduction of lateral load capacity by more than 5% in any story; d) earthquake damage reducing lateral load capacity by more than 10% at any story.

The federal government likewise, in RP 6, [NIST, 2002], also has definitive triggers: a) change in occupancy that increases the building's importance or level of use; b) alteration cost exceeding 50% of replacement cost; c) damage of any kind that has significantly degraded the lateral system; d) deemed to be a high seismic risk; and e) added to federal inventory though purchase or donation.

The regulations and policies governing any building, private or public, which will be significantly altered, should be researched in the planning stage to understand the effective seismic triggers, written or understood.

(Unreinforced masonry, for example, was not only prohibited as a structural system in zones of high seismicity, but also could not be used in a building at all.) Use of a lateral force level of 75% of that required for new buildings became fairly standard, but the treatment of archaic materials is highly variable from jurisdiction to jurisdiction.

Many local retrofit provisions are gradually being replaced by national guidelines and standards for seismic evaluation and retrofit (e.g. ASCE 31, 2003; FEMA 356, 2000, etc.). In addition, performance based seismic design is enabling a more direct approach to meeting a community's minimum performance standards—although this requires the policy-makers to decide what the minimum performance standard should be, a difficult task that crosses social, economic, and technical boundaries.

In summary, both the passive triggers for seismic retrofit and the design or performance criteria are often ill-defined and, at best, highly variable between jurisdictions. Design professionals should always determine the governing local, state, or federal regulations or policies when designing alterations or remodels on existing buildings.

● Active Code Provisions

Active code provisions result from policy decisions of a jurisdiction to reduce the community seismic risk by requiring seismic upgrading of certain buildings known to be particularly vulnerable to unacceptable damage. For the most part, these provisions are unfunded mandates, although low-interest loan programs have been developed in some cases. These risk reduction programs usually allow owners a lengthy period to perform the retrofit or to demolish the buildings—ten years or more. The standard for retrofit is also normally included in the law or regulation and is typically prescriptive, although performance-based design options are becoming more acceptable.

Two large-scale examples of active seismic code provisions were started by the state of California. The first was a program to reduce the risk from URM buildings. The state legislature, lacking the votes to simply require mitigation throughout the state, instead passed a law (SB 547-1986) that required local jurisdictions to develop inventories of these buildings in their area, to notify the owners that their building was considered hazardous, and to develop a community-wide hazard reduction plan.

Although not required to do so, most jurisdictions chose as their hazard reduction plan to pass active code ordinances giving owners of the buildings ten or so years to retrofit them. Over 10,000 URM buildings have been brought into compliance with these local ordinances, most by retrofit, but some by demolition (SSC, 2003).

The second program, created by SB 1953 in 1994 following the Northridge earthquake, gave California hospital owners until 2030 to upgrade or replace their hospitals to comply with state law governing new hospital buildings. The program's intention is to enable buildings to be functional following an earthquake. This law affected over 500 hospitals and over 2,000 buildings (Holmes, 2002). Although compliance is ongoing, this law has been problematic due to the high cost and disruption associated with retrofitting hospital buildings, and the highly variable economic condition of the health system as well as individual facilities.

Other examples include local ordinances to retrofit tilt-up buildings, less controversial because of the clear high vulnerability and low retrofit cost of these buildings. Similar to investigating local regulations regarding triggers, it is also wise to determine if any existing building planned for alterations is covered by (or will be covered in the foreseeable future) a requirement to retrofit. It is generally acknowledged that seismic improvements are easier to implement when done in association with other work on the building.

● Post-Earthquake Code Provisions

Following a damaging earthquake, many buildings may be closed pending determination of safety and necessary repairs. A lack of clear repair standards and criteria for re-occupancy has created controversy and denied owners use of their buildings after most damaging earthquakes. Assuming that the earthquake itself is the ultimate judge of seismic acceptability, many communities may take the opportunity in the post-earthquake period to require strengthening of buildings that are apparently seismically deficient due to their damage level. However, implementation of this theory incorporating conservative policies that require many retrofits may delay the economic recovery of the community. On the other hand, standards for repair and/or strengthening which are not conservative could lead to equal or worse damage in the next earthquake. It has also been observed that owners of historic, rent-controlled, or otherwise economically controlled buildings may have an incentive to demolish damaged buildings to the detriment of the community at large.

Traditionally, communities (building departments) have used color codes for several or all of the following categories of buildings following an earthquake:

A. Undamaged; no action required. If inspected at all, these buildings will be Green-tagged.

B. Damaged to a slight extent that will only require repair of the damage to the original condition. These buildings will generally be Green-tagged, but the category could also include some Yellow-tags.

C. Damaged to a greater extent that suggests such seismic weaknesses in the building that the overall building should be checked for compliance with minimum seismic standards. This will often require overall retrofit of the building. These buildings will generally be Red-tagged, but the category could also include some Yellow tags.

C1. (A subcategory of C). Damaged to an extent that the building creates a public risk that requires immediate mitigation, either temporary shoring or demolition. The ultimate disposition of these buildings may not be determined for several months. These buildings will all be Red-tagged.

The most significant categorization is the differentiation between B and C. The difference to an owner between being placed in one category or the other could be an expense on the order of 30%-50% of the value of the building, reflecting the added cost of retrofit to that of repair. Earthquakes being rare, few communities have been forced to create these policies, but a few have. Oakland, California, prior to the Loma Prieta earthquake, set a trigger based on the loss of capacity caused by the damage. If the damage was determined to have caused a loss of over 10% of lateral force capacity, then retrofit was triggered. Los Angeles and other southern California communities affected by the Northridge earthquake used a similar standard, but the 10% loss was applied to lines of seismic resistance rather than the building as a whole. These code regulations, although definitive, are problematic because of the technical difficulty of determining loss of capacity, particularly to the accuracy of 1% (a 1% change can trigger a retrofit).

The importance of this issue has been magnified by interpretation of federal laws that creates a tie between reimbursement of the cost of repair of certain local damage to the pre-existence and nature of these local damage triggers. Owners and designers of older existing buildings should be aware of such triggers that could affect them should they suffer damage from an earthquake. In some cases, it may be prudent for an owner to voluntarily retrofit a vulnerable building to avoid the possibility of being forced to do it in a post-earthquake environment as well as to possibly avoiding a long closure of the building.

8.3 THE FEMA PROGRAM TO REDUCE THE SEISMIC RISK FROM EXISTING BUILDINGS

In 1985, the Federal Emergency Management Agency (FEMA) recognized that the principal seismic risk in this country came from the existing building stock, the majority of which was designed without adequate seismic provisions. Following a national workshop that identified significant issues and potential educational and guideline projects that FEMA could lead, a program was launched that is still ongoing. In addition to providing education and technical guidelines in the area of high-risk existing buildings, other FEMA programs were also significant in enabling communities to understand and mitigate their seismic risk, most notably the development of the regional loss-estimating computer program, HAZUS. Most of these activities are documented as part of the FEMA "yellow book" series (so known because of its distinctive yellow covers), well known to engineers in this country and, in fact, around the world. Unfortunately, these documents are less known to architects, although many of them contain useful insights into not only the issues surrounding seismic evaluation and retrofit of existing buildings, but also into all aspects of seismic design.

8.3.1 FEMA-Sponsored Activity for Existing Buildings

Following is a summary of selected FEMA-sponsored projects beginning in the late 1980s. A full listing is given in FEMA 315, *Seismic Rehabilitation of Buildings, Strategic Plan 2005*.

- Rapid Visual Screening

  FEMA 154: *Rapid Visual Screening of Buildings for Potential Seismic Hazards*, 1988, updated 2001

  A method to enable an efficient first sorting of selected buildings into an adequately life-safe group and a second group that will require further evaluation. The evaluation was intended to be performed on the street in an hour or less per building. The first task is to assign the building to a predefined model building type and then identify additional characteristics that could refine the seismic vulnerability. The method has proven useful to efficiently generate an approximate mix of buildings that will properly characterize a community's vulnerability, but not to definitely rate individual buildings, due to the difficulty of identifying significant features from the street. Generally it is necessary to obtain access to the interior of a building, or, more commonly, it is even necessary to review drawings to confidently eliminate older buildings as potentially hazardous.

- ## Evaluation of Existing Buildings

FEMA 178: *NEHRP Handbook for Seismic Evaluation of Existing Buildings*, 1989 and FEMA 310: *Handbook for the Seismic Rehabilitation of Buildings: a Prestandard.*

A widely used guide to determine if individual buildings meet a nationally accepted level of seismic life safety. This method requires engineering calculations and is essentially prescriptive, which facilitates consistency and enables enforceability. This life-safety standard was adopted by, among others, the federal government and the state of California in certain programs. The prescriptive bar may, however, have been set too high, because very few older buildings pass. Since its original development, FEMA 178 has been refined and republished as FEMA 310, and finally was adopted as a Standard by the American Society of Civil Engineers as ASCE 31 in 2003.

- ## Techniques Used in Seismic Retrofit

FEMA 172: *NEHRP Handbook of Techniques for Seismic Rehabilitation of Existing Buildings*, 1992

In recognition of the lack of experience in seismic upgrading of buildings of most of the country's engineers and architects, this document outlined the basic methods of seismically strengthening a building, including conceptual details of typically added structural elements. The material recognizes the FEMA model building types, but is primarily organized around strengthening of structural components, perhaps making the material less directly accessible. The document also preceded by several years the publication of the analytical tools to design retrofits (FEMA 273, see below). For whatever reason, the publication went relatively unused, despite the fact that it contains useful information, particularly for architects unfamiliar with seismic issues.

Financial Incentives

FEMA 198: *Financial Incentives for Seismic Rehabilitation of Hazardous Buildings*, 1990

To encourage voluntary seismic upgrading, this document described the financial incentives to do so, ranging from tax benefits to damage avoidance.

Development of Benefit-Cost Model

FEMA 227: *A Benefit-Cost Model for the Seismic Rehabilitation of Buildings*, 1992

Due to the expected high cost of seismic rehabilitation, the need to provide a method to calculate the benefit-cost ratio of seismic retrofit was identified early in the program. This requires estimation of financial losses from earthquake damage resulting from a full range of ground-shaking intensity. Financial losses include direct damage to structural and nonstructural systems as well as business interruption costs. A controversial feature was the optional inclusion of the value of lives lost in the overall equation. The project was primarily to develop the model rather than to provide new research into expected damage or casualty rates. Thus, approximate relationships available at the time were used. However, the documentation concerning contributing factors to a benefit-cost analysis is quite complete, and a computerized functional spreadsheet version of the method was developed.

Although the use of benefit-cost analysis never became popular in the private sector, trials of the method indicated that very low retrofit costs, high business-interruption losses, or high exposure-to-casualty losses are required to result in a positive benefit-cost ratio. For example rehabilitation of tilt-up buildings is usually fairly inexpensive and usually proves cost effective. Similarly, buildings with high importance to a business or with high occupancy in areas of high seismicity also result in positive results. Despite the apparent overall results from this program, considerable rehabilitation activity continued, both in conditions expected to yield a positive benefit-cost and in other conditions (in many cases due to extreme importance given to life safety).

Typical Costs of Seismic Rehabilitation

FEMA 156: *Typical Costs for Seismic Rehabilitation of Existing Buildings,* 1988, 2nd edition, 1994.

The second edition of this document collected case histories of constructed rehabilitation and completed reports judged to have realistic costs. A database was created to separate costs by primary influence factors: model building types, rehabilitation performance objectives, and seismicity. A serious difficulty in collection of accurate data was the inevitable mixing of pure rehabilitation costs and associated costs such as life-safety upgrades, the American Disabilities Act (ADA), and even remodels. Although a large amount of data was collected, there was not nearly enough to populate all combinations of the factors. Nevertheless, a method was developed to use the data to make estimates of costs for given situations. The major problem was that the coefficient of variation of rehabilitation costs, for any given situation, is very high due to high variability in the extent of seismic deficiencies. The information collected is probably most useful to estimate costs for large numbers of similar buildings where variations will average out. Use of the method to accurately estimate the cost of a single building is not recommended, although even the ranges given could be useful for architects and engineers not familiar with retrofit issues.

Technical Guidelines for Seismic Rehabilitation

FEMA 273: *NEHRP Guidelines for the Seismic Rehabilitation of Buildings,* 1997 and FEMA 356 *Prestandard and Commentary for the Seismic Rehabilitation of Buildings.*

This document, developed over five years by over 70 experts, was the culmination of the original program. Previously, the most common complaint from engineers and building officials was the lack of criteria for seismic retrofit. FEMA 273 incorporated performance-based engineering, state-of-the-art nonlinear analysis techniques, and an extensive commentary to make a significant contribution to earthquake engineering and to focus laboratory research on development of missing data. The document broke away from traditional code methods and in doing so, faced problems of inconsistency with the design of new buildings. Improvements were made in a follow-up document, FEMA 356, but the practical results from use of the method indicate that considerable judgment is needed in application. Work is continuing to improve analysis methods and in methods to predict the damage level to both components of a building and to the building as a whole. Even given these difficulties, the document has become the standard of the industry.

Development of a Standardized Regional Loss Estimation Methodology—HAZUS

National Institute of Building Sciences, *Earthquake Loss Estimation Methodology, Technical Manual* (for latest release).

A good summary of this development is contained in a paper by Whitman, et al. in EERI *Earthquake Spectra*, vol 13, no 4. FEMA also maintains a HAZUS website, www.fema.gov/hazus.

The development of a standardized regional loss estimation methodology was not in the original FEMA plan to reduce risks in existing buildings. However, this development has had a major impact in educating local officials about their seismic risks and estimating the level of risk around the country in a standard and comparable way.

In 1990, when the development of HAZUS began, the primary goals were to raise awareness of potential local earthquake risks, to provide local emergency responders with reasonable descriptions of post-earthquake conditions for planning purposes, and to provide consistently created loss estimates in various regions to allow valid comparison and analysis. The loss estimation methodology was intended to be comprehensive and cover not only building losses, but also damage to transportation systems, ports, utilities, and critical facilities. A technically defensible methodology was the goal, not necessarily an all-encompassing software package. When it became obvious that the methodology was far more useful and could be more consistently applied as software, HAZUS was born. The program uses census data and other available physical and economic databases to develop, on a first level of accuracy, a model of local conditions. Expected or speculated seismic events can be run and losses estimated. Losses include direct damage, business interruption, and casualties, as well as loss of utilities, loss of housing units, and many other parameters of use to emergency planners. The building inventory uses the FEMA model building types and an analysis method closely tied to FEMA 273, linking HAZUS to other FEMA-sponsored work regarding existing buildings.

Subsequent to the original development activity, HAZUS was expanded to create loss estimates for wind and flood.

- Incremental Rehabilitation

FEMA 395, *Incremental Seismic Rehabilitation of School Buildings (K-12)*, 2003.

This is the first in a series of manuals that FEMA (U.S. Department of Homeland Security) intends to develop for various occupancy types including, for example, schools, hospitals, and office buildings. The concept is based on the fact that seismic strengthening activities are more efficiently accomplished in conjunction with other work on the building, and such opportunities should be identified and exploited even if only part of a complete rehabilitation is accomplished. This is perhaps most applicable to K-12 school buildings because of their relatively small size and ongoing maintenance programs. FEMA model building types are again used to categorize potential opportunities in different conditions. As is pointed out in the manual, this technique has to be applied with care to avoid an intermediate structural condition that is worse than the original.

8.3.2 The FEMA Model Building Types

Most of these developments were part of the integrated plan developed in 1985. As such, FEMA coordinated the projects and required common terminology and cross-references.

The most successful and virtually standard-setting effort was the creation of a set of model building types to be used for the characterization of existing buildings. The model building types are based primarily on structural systems rather than occupancy, but have proven extremely useful in the overall program. Model building types are defined by a combination of the gravity-load carrying system and the lateral-load carrying system of the building. Not every building type ever built in the country and certainly not the world is represented, but the significant ones are, and the relative risks of a community can well be represented by separating the local inventory into these types. Of course, there was no attempt to represent every "modern" building type because they are not considered hazardous buildings. However, with minor sub-categorization that has occurred with successive documents, the majority of buildings, new or old, now can be assigned a model building type. The test of the usefulness came with the successful development of HAZUS using the model building type because this program needed a reasonably simple method to characterize the seismic vulnerability of inventories of buildings across the country.

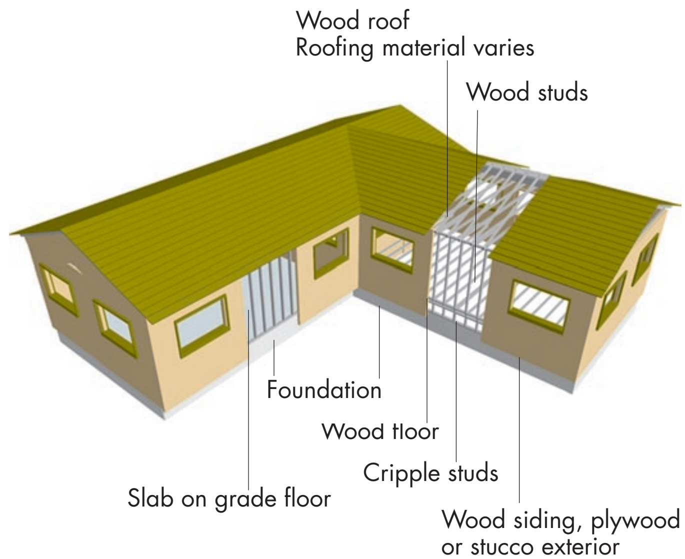
Currently, no single FEMA document contains a graphic and clear description of the model building types, although engineers can generally determine the correct category. Because of the ubiquitous FEMA-developed documents, guidelines, and standards regarding existing buildings, and their common use by engineers, such descriptions are included here to facilitate communication with architects. The types are illustrated on pages 8-23 through 8-31. Table 8-3, at the end of the chapter, presents a summary of the performance charactoristics and commom rehabilitation techniques.

8.4 SEISMIC EVALUATION OF EXISTING BUILDINGS

Not all older buildings are seismically at risk. If they were, the damage from several earthquakes in this country, including the 1971 San Fernando and the 1984 Northridge events, would have been devastating, because much of the inventory affected was twenty or more years old. Often, strong ground shaking from earthquakes significantly damages building types and configurations well known to be vulnerable, and occasionally highlights vulnerabilities previously unrealized. For example, the Northridge earthquake caused damage to many wood-frame buildings—mostly apartments—and relatively modern steel moment-frame buildings, both previously considered to be of low vulnerability. It is natural to catalogue damage after an earthquake by buildings with common characteristics, the most obvious characteristic being the construction type, and the secondary characteristic being the configuration. Both of these parameters are central to processes developed to identify buildings especially vulnerable to damage before the earthquake. In fact, the categorization of damage by building type is primarily what led to the development of the FEMA Model Building Types discussed in Section 8.3.2.

However, only in the most vulnerable building types does damage occur relatively consistently. For example, at higher levels of shaking, the exterior walls of unreinforced masonry bearing-wall buildings have relatively consistently fallen away from their buildings in many earthquakes, ever since this building type was built in large numbers in the late 19th century. More recently, a high percentage of "pre-Northridge" steel

FEMA Building Type W1 WOOD LIGHT FRAME (small residence)

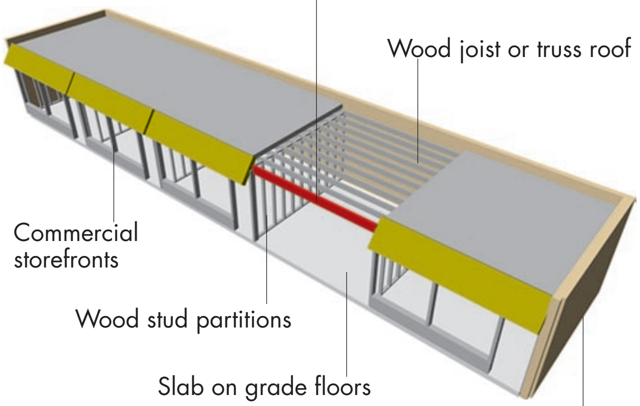


These buildings are generally single-family dwellings of one and two stories. Floor and roof framing consists of wood joists or rafters supported on wood studs spaced no more than 24 inches apart. The first floor may be slab on grade or wood raised above grade with cripple stud walls and post-and-beam supports. Lateral support is provided with shear walls of plywood, stucco, gypsum board, and a variety of other materials. Most often there is no engineering design for lateral forces.

FEMA Building Type W1A WOOD LIGHT FRAME (multi-unit residence)

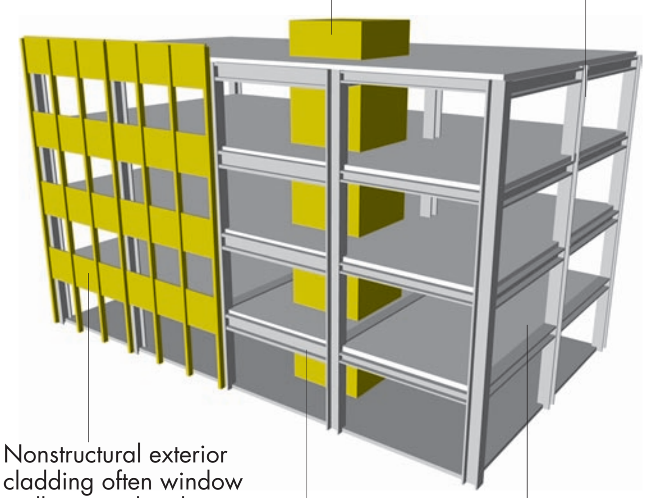
These buildings are framed with the same systems as W1 buildings but are most often multiple-story large residential-type structures, and, unless very old, are engineered. A common seismic deficiency is the tuck-under parking at the ground story that creates a soft or weak story. This building type is also often built on top of a one story concrete parking structure.

FEMA Building Type W2 WOOD FRAME (commercial and industrial)



These buildings are commonly commercial or smaller industrial buildings and are constructed primarily of wood framing. The floor and roof framing consists of wood joists and wood or steel trusses, glulam or steel beams, and wood posts or steel columns. Lateral forces are resisted by wood diaphragms and exterior stud walls sheathed with plywood, stucco, or wood sheathing, or sometimes rod bracing or a spot steel-braced frame. Large wall openings are common for storefronts or garage openings. This building type is also often used for schools, churchs and clubhouses.

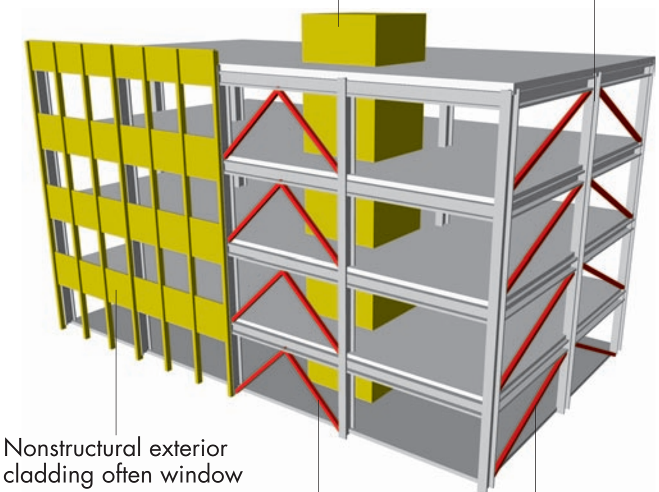
FEMA Building Type S1 STEEL MOMENT FRAMES



These buildings consist of an essentially complete frame assembly of steel beams and columns. Lateral forces are resisted by moment frames that develop stiffness through rigid connections of the beam and column created by angles, plates and bolts, or by welding. Moment frames may be developed on all framing lines or only in selected bays. It is significant that no structural walls are required. Floors are cast-in-place concrete slabs or metal deck and concrete. This building is used for a wide variety of occupancies such as offices, hospitals, laboratories, and academic and government buildings.

The S1A building type is similar but has floors and roof that act as flexible diaphragms, such as wood or uptopped metal deck. One family of these buildings are older warehouse or industrial buildings, while another more recent use is for small office or commercial buildings in which the fire rating of concrete floors is not needed.

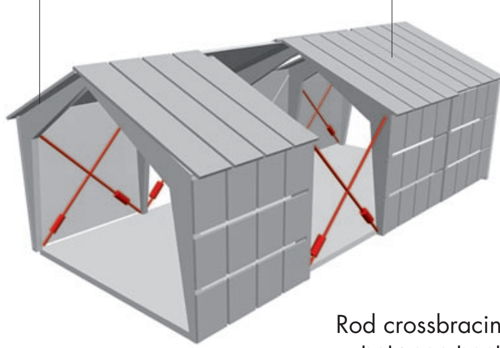
FEMA Building Type S2 STEEL-BRACED FRAMES



These buildings consist of a frame assembly of steel columns and beams. Lateral forces are resisted by diagonal steel members placed in selected bays. Floors are cast-in-place concrete slabs or metal deck and concrete. These buildings are typically used for buildings similar tos teel-moment frames, although are more often low rise.

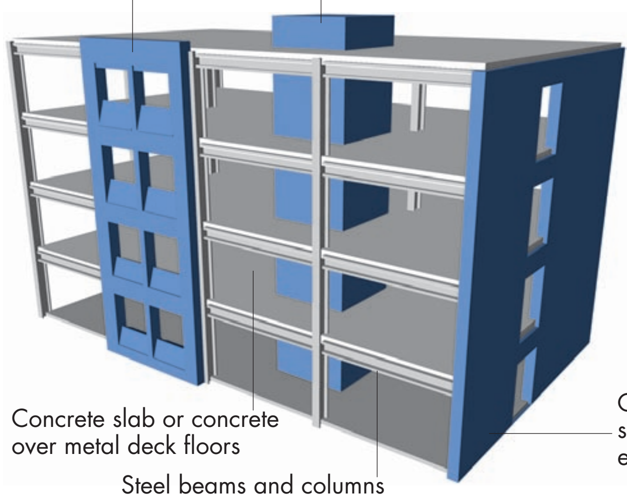
The S2A building type is similar but has floors and roof that act as flexible diaphragms such as wood, or uptopped metal deck. This is a relatively uncommon building type and is used mostly for smaller office or commercial buildings in which the fire rating of concrete floor is not needed.

FEMA Building Type S3 STEEL LIGHT FRAMES



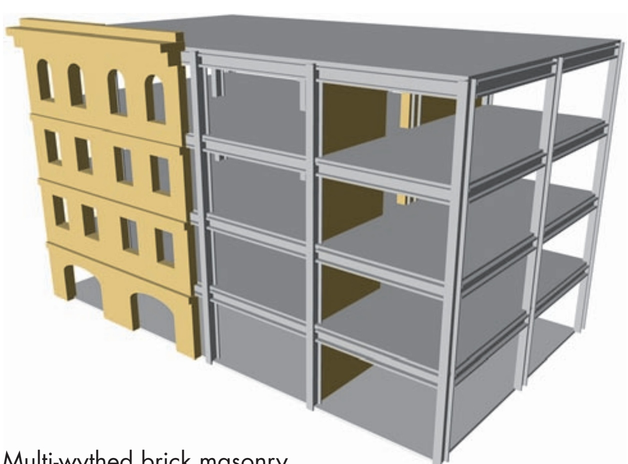
These buildings are one story, pre-engineered and partially prefabricated, and normally consist of transverse steel bents and light purlins. The roof and walls consist of lightweight metal, fiberglass, or cementitious panels. Lateral forces are resisted by the transverse steel bents acting as moment frames, and light rod diagonal bracing in the longitudinal direction. The roof diaphragm is either metal deck or diagonal rod bracing. These buildings are mostly used for industrial or agricultural occupancies.

FEMA Building Type S4 STEEL FRAMES with concrete shearwalls



These buildings consist of an essentially complete frame assembly of steel beams and steel columns. The floors are concrete slabs or concrete fill over metal deck. The buildings feature a significant number of concrete walls effectively acting as shear walls, either as vertical transportation cores, isolated in selected bays, or as a perimeter wall system. The steel column-and-beam system may act only to carry gravity loads or may have rigid connections to act as a moment frame. This building type is generally used as an alternate for steel moment or braced frames in similar circumstances. These buildings will usually be mid- or low-rise.

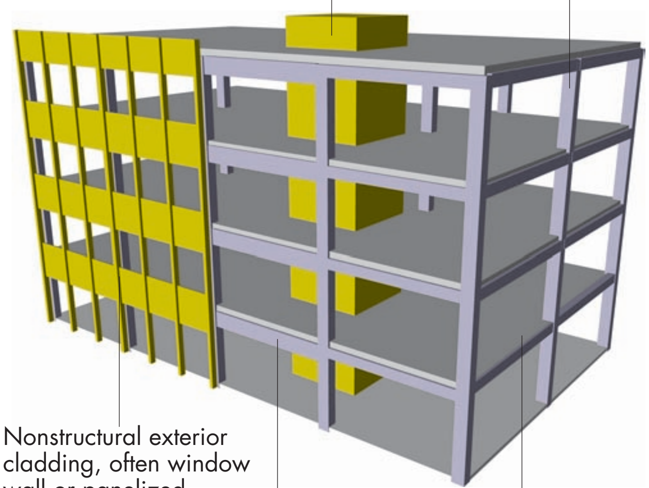
FEMA Building Type S5 STEEL FRAMES with infill masonry walls



This is normally an older building that consists of an essentially complete frame assembly of steel floor beams or trusses and steel columns. The floor consists of masonry flat arches, concrete slabs or metal deck, and concrete fill. Exterior walls and possibly some interior walls, are constructed of unreinforced solid clay brick, concrete block, or hollow-clay tile masonry infilling the space between columns and beams. Windows and doors may be present in the infill walls, but to act effectively as shear-resisting elements, the infill masonry must be constructed tightly against the columns and beams. Although relatively modern buildings in moderate or low seismic regions are built with unreinforced masonry exterior infill walls, the walls are generally not built tight against the beams and columns and therefore do not provide shear resistance. The buildings intended to fall into this category feature exposed clay brick masonry on the exterior and are common in commercial areas of cities with occupancies of retail stores, small offices, and hotels.

The S5A building type is similar but has floors and roof that act as flexible diaphragms, such as wood or uptopped metal deck. These buildings will almost all date to the 1930s and earlier, and were originally warehouses or industrial buildings.

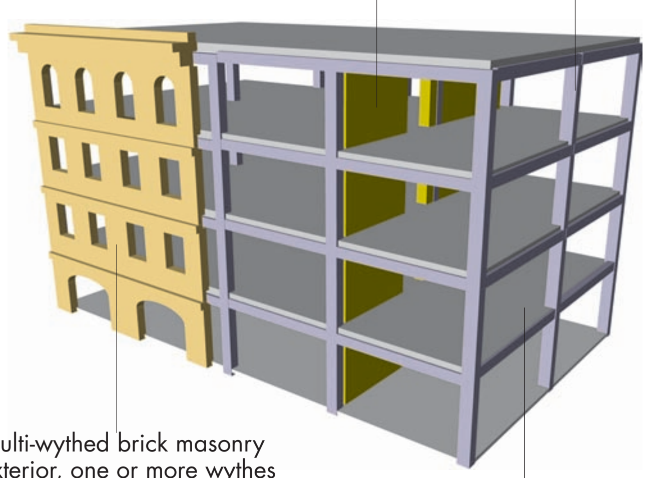
FEMA Building Type C1 CONCRETE MOMENT FRAMES



These buildings consist of concrete framing, either a complete system of beams and columns or columns supporting slabs without gravity beams. Lateral forces are resisted by moment frames that develop stiffness through rigid connections of the column and beams placed in a given bay. Moment frames may be developed on all framing lines or only in selected bays. It is significant that no structural walls are required. Floors are cast-in-place or precast concrete. Buildings with concrete moment frames could be used for most occupancies listed for steel moment frames, but are also used for multistory residential buildings.

The C1A building type is similar but has floors and roof that act as flexible diaphragms, such as wood or uptopped metal deck. This is a relatively unusual building type, but might be found as older warehouse-type buildings or small office occupancies.

FEMA Building Type C3 CONCRETE FRAMES with infill masonry shear walls

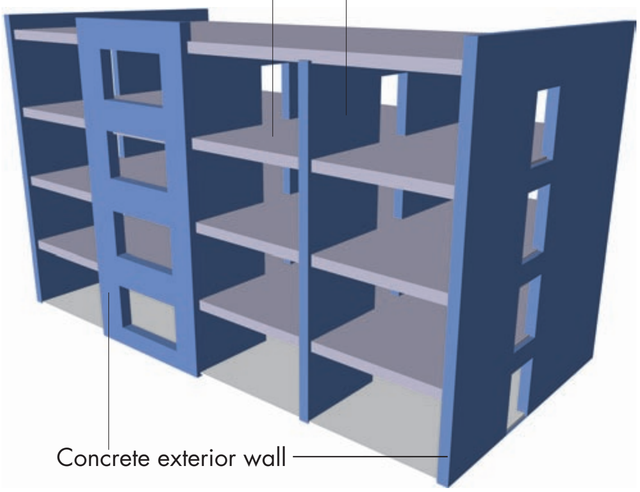


These buildings consist of concrete framing, either a complete system of beams and columns or columns supporting slabs without gravity beams. Exterior walls and possibly some interior walls are constructed of unreinforced solid clay brick, concrete block, or hollow clay tile masonry infilling the space between columns and beams. Windows and doors may be present in the infill walls, but to act effectively as shear-resisting elements, the infill masonry must be constructed tightly against the columns and beams. The building type is similar to S5, but is more often used for industrial and warehouse occupancies.

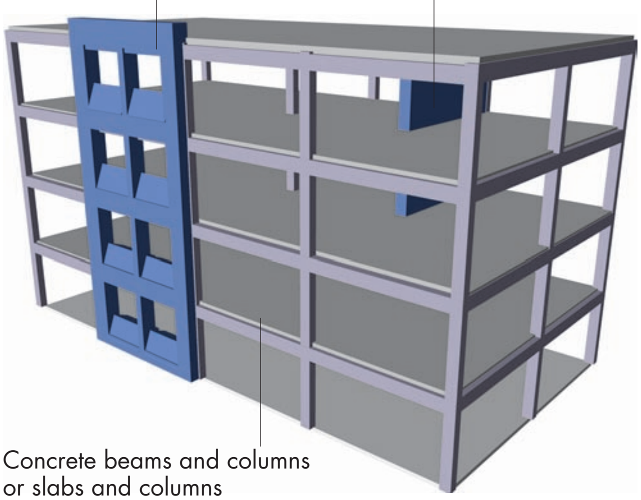
The C3A building type is similar but has floors and roof that act as flexible diaphragms, such as wood, or uptopped metal deck. Thisbuilding type not often found except as one-story industrial buildings.

FEMA Building Type C2 CONCRETE SHEAR WALLS

with bearing walls



with gravity frames



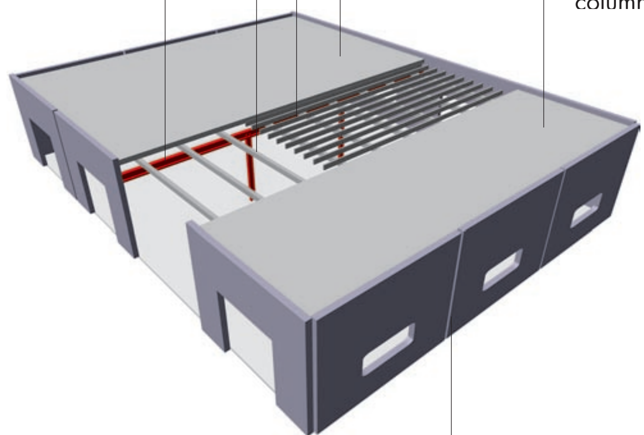
Concrete shear walls are concrete walls in a building design to provide lateral stiffness and strength for lateral loads. There are two main types of shear-wall buildings, those in which the shear walls also carry the gravity loads (with bearing walls), and those in which a column-supported framing system carries the gravity loads (with gravity frame).

In the **bearing wall** type, all walls usually act as both bearing and shear walls. The building type is similar and often used in the same occupancies as type RM2, namely in mid- and low-rise hotels and motels. This building type is also used in residential apartment/condo-type buildings.

In **gravity frame** buildings, shear walls are either strategically placed around the plan, or at the perimeter. Shear-wall systems placed around the entire perimeter must contain the windows, and other perimeter openings are called punched shear walls. These buildings were commonly built in the 1950s and 1960s for a wide variety of most institutional occupancy types.

The C2A building type is similar, but has floors and roof that act as flexible diaphragms such as wood, or uptopped metal deck. C2A buildings are normally bearing-wall buildings. These buildings are similar to building-type RM1 and are used for similar occupancies- such as small office or commercial and sometimes residential.

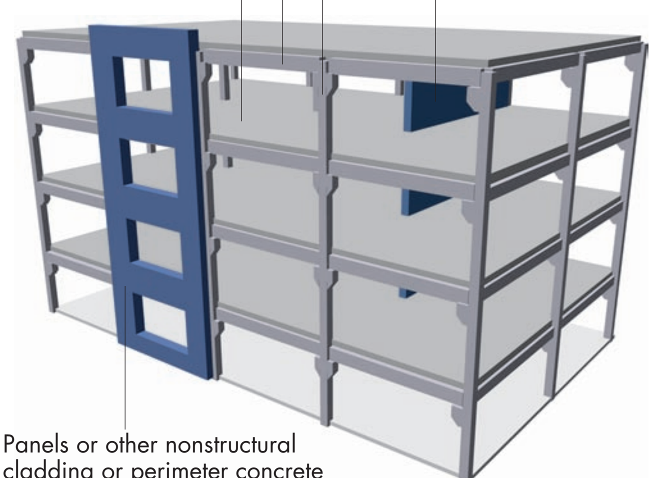
FEMA Building Type PC1 TILT-UP CONCRETE shear walls



These buildings are constructed with perimeter concrete walls precast on the site and tilted up to form the exterior of the buildings, to support all or a portion of the perimeter roof load, and to provide seismic shear resistance. These buildings are commonly one-story with a wood joist and plywood roof or sometimes with a roof of steel joists and metal deck. Two-story tilt-ups usually have a steel-framed second floor with metal deck and concrete and a wood roof. Tilt-up walls that support roof load are very common on the West Coast; due to economical construction cost, they are used for many occupancies, including warehouses, retail stores, and offices. In other parts of the country, these buildings more often have an independent load-carrying system on the inside face of the walls.

The PC1A building is similar but features all floors and/or roof constructed of materials that form a rigid diaphragm, normally concrete. This building type is similar to PC2.

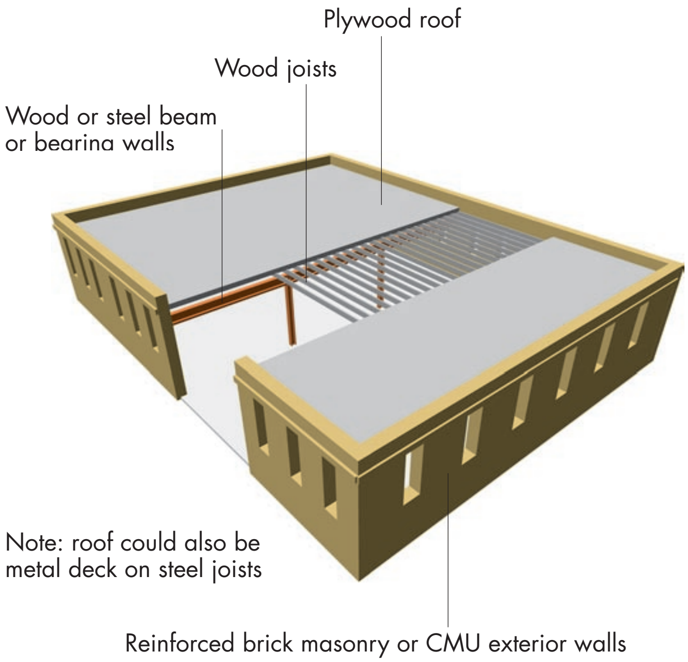
FEMA Building Type PC2 PRECAST CONCRETE FRAMES with shear walls



These buildings consist of concrete columns, girders, beams and/or slabs that are precast off the site and erected to form a complete gravity-load system. Type PC2 has a lateral force-resisting system of concrete shear walls, usually cast-in-place. Many garages have been built with this system The building type is most common in moderate and low seismic zones and could be used for many different occupancies in those areas.

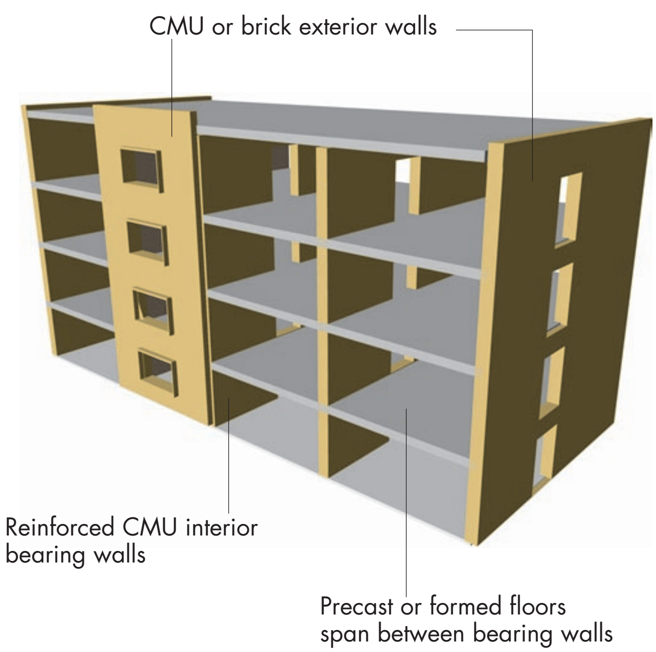
The PC2A building is similar but obtains lateral support from specially connected precast girders and columns that form moment frames. Until recently, precast moment frames have not been allowed in regions of high seismicity, and these buildings will essentially only be found in moderate or low seismic zones.

FEMA Building Type RM1 REINFORCED MASONRY WALLS with flexible diaphrams



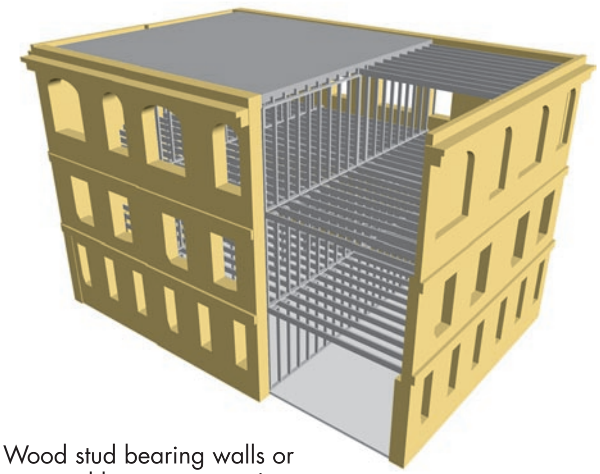
These buildings take a variety of configurations, but they are characterized by reinforced masonry walls (brick cavity wall or CMU) with flexible diaphragms, such as wood or metal deck. The walls are commonly bearing, but the gravity system often also contains post-and-beam construction of wood or steel. Older buildings of this type are generally small and were used for a wide variety of occupancies and are configured to suit. Recently, the building type is commonly used for one-story warehouse-type occupancies similar to tilt-up buildings.

FEMA Building Type RM2 REINFORCED MASONRY WALLS with stiff diaphrams



This building consists of reinforced masonry walls and concrete slab floors that may be either cast-in-place or precast. This building type is often used for hotel and motels and is similar to the concrete bearing-wall type C2.

FEMA Building Type URM UNREINFORCED MASONRY BEARING WALLS



This building consists of unreinforced masonry bearing walls, usually at the perimeter and usually brick masonry. The floors are wood joists and wood sheathing supported on the walls and on interior post-and-beam construction or wood-stud bearing walls. This building type is ubiquitous in the U.S. and was built for a wide variety of uses, from one-story commercial or industrial occupancies, to multistory warehouses, to mid-rise hotels. Unfortunately, it has consistently performed poorly in earthquakes. The most common failure is an outward collapse of the exterior walls, caused by loss of lateral support due to separation of the walls from the floor/roof diaphragm.

The URMA building is similar. but features all floors and/or roof constructed of materials that form a rigid diaphragm, usually concrete slabs or steel joists with flat-arched unreinforced masonry.

moment-frame buildings have received damage to their beam-column connections when subjected to strong shaking. Even in these cases, the damage is not 100% consistent and certainly not 100% predictable. In building types with less vulnerability, the damage has an even higher coefficient of variation. Engineers and policymakers, therefore, have struggled with methods to reliably evaluate existing buildings for their seismic vulnerability.

As discussed in Section 8.2, the initial engineering response was to judge older buildings by their capacity to meet the code for new buildings, but it became quickly apparent that this method was overly conservative, because almost every building older than one or two code-change cycles would not comply—and thus be considered deficient. Even when lower lateral force levels were used, and the presence of archaic material was not, in itself, considered a deficiency, many more buildings were found deficient than was evidenced in serious earthquake damage. Thus, policymakers have generally been successful in passing active retrofit provisions (see Section 8.2.3) only in the most vulnerable buildings, such as URM and tilt-ups, where damage has been significant and consistent, and individual building evaluation is not particularly significant.

The evaluation of existing buildings typically starts with identification of the building type and damaging characteristics of configuration (e.g., soft story). This can be done rapidly and inexpensively but, except for a few vulnerable building types, is unreliable when taken to the individual building level. Engineers and code writers have also developed intermediate levels of evaluation in which more characteristics are identified and evaluated, many by calculation. In the last decade, more sophisticated methods of analysis and evaluation have been developed that consider the nonlinear response of most structures to earthquakes and very detailed material and configuration properties that will vary from building to building.

8.4.1 Expected Performance by Building Type

As previously mentioned, damage levels after earthquakes are collected and generally assigned to bins of common characteristics, most commonly the level of shaking, building material and type, and configuration. Combined with numerical lateral-force analysis of prototype buildings, this information can be analyzed statistically. The three primary parameters - building type, shaking level, and damage level - are often displayed together in a damage probability matrix similar to Table 8-1. The variability of damage is such that for any shaking level, as shown in the columns, there is normally a probability that some buildings will be in each damage state. The probabilities in these tables can be interpreted as the percentage of a large number of buildings expected to be in each damage state, or the chances, given the shaking level, that an individual building of this type with be damaged to each level.

Statistical information such as this is used in several ways:

- Identification of clearly vulnerable or dangerous buildings to help establish policies of mitigation

Many extremely vulnerable building types or components can be identified by observation without statistical analysis, including URM, soft-story "tuck-under" apartment buildings, the roof-to-wall connection in tilt-up

Table 8-1: Typical Form of Damage Probability Matrix

Damage Level	Strength of Ground Motion (Peak Ground Acceleration, Spectral Acceleration, or Modified Mercalli Intensity [MMI]. MMI shown here)				
	VI	VII	VIII	IX	X
None	.84	.65	.05	.02	-
Slight	.15	.28	.75	.31	.20
Moderate	.01	.03	.10	.47	.50
Severe	-	.03	.08	.15	.20
Complete	-	.01	.02	.05	.10

buildings, residences with cripple wall first-floor construction, and connections of pre-Northridge steel moment-frames. The clearly and more consistently dangerous building types have often generated enough community concern to cause the creation of policies to mitigate the risks with retrofit. For a combination of reasons, URMs and tilt-ups currently are the targets of the most active mitigation policies.

● Earthquake Loss Estimation

Regional earthquake loss estimates have been performed for forty or more years to raise awareness in the community about the risks from earthquakes and to facilitate emergency planning. Given an approximate distribution of the building inventory and a map of estimated ground motion from a given earthquake, damage-probability matrices (or similar data) can be used to estimate damage levels to the building stock. From the damage levels, economic loss, potential casualties, and business interruption in a community can be estimated.

Starting in 1991, FEMA began a major program to develop a standard way of performing such loss estimations to facilitate comparative loss estimates in various parts of the country. This program resulted in a computer program, HAZUS, described briefly in Section 8.3.1.

● Formal Economic Loss Evaluations (e.g. Probable Maximum Loss or PML)

Since consensus loss relationships became available (ATC, 1985), a demand has grown to include an estimate of seismic loss in "due-diligence" studies done for purchase of buildings, for obtaining loans for purchase or refinance, or for insurance purposes. An economic loss parameter, called Probable Maximum Loss, has become the standard measuring stick for these purposes. The PML for a building is the pessimistic loss (the loss suffered by the worst 10% of similar buildings) for the worst shaking expected at the site (which gradually became defined as the shaking with a 500-year return period, similar to the code design event). Although a detailed analysis can be performed to obtain a PML, most are established by building type and a few observable building characteristics. Because of the high variability in damage and the relatively incomplete statistics available, PMLs are not very reliable, particularly for an individual building.

- Rapid Evaluation

As foreseen by FEMA's original plan for the mitigation of risks from existing buildings, a rapid evaluation technique should be available to quickly sort the buildings into three categories: obviously hazardous, obviously acceptable, and uncertain. The intent was to spend less than two hours per building for this rapid evaluation. Under the plan, the uncertain group would then be evaluated by more detailed methods. The results of FEMA's development efforts, FEMA 154 (Section 8.3.1) is fairly sophisticated but, because of the large amount of unknown building data that is inherent in the system, for an individual building, is unreliable. The sorting method is probably quite good for estimating the overall vulnerability of a community because of the averaging effect when estimating the risk of many buildings.

8.4.2 Evaluation of Individual Buildings

Engineers have been seismically evaluating existing buildings for many years, whether by comparing the conditions with those required by the code for new buildings, by using some local or building-specific standard (e.g. URMs), or by using their own judgment. These methods are still used, as well as very sophisticated proprietary methods developed within private offices, but the majority of evaluations are now tied in some way to the general procedures of ASCE 31-03, *Seismic Evaluation of Existing Buildings* (ASCE, 2003), that in 2003 became a national standard. There are three levels of evaluation in the standard called tiers, which, not accidentally, are similar to the standard of practice prior to the standardization process. These levels of evaluation are briefly described below, as well as similar methods that fall in the same categories.

However, before beginning a seismic evaluation, particularly of a group of buildings, it is logical to assume that buildings built to modern codes must meet some acceptable standard of life safety. Due to the large number of older buildings, the effort to eliminate some from consideration resulted in several well-known milestone years. First it was compliance with the 1973 *Uniform Building Code* (UBC) or equivalent. After study and reconsideration of relatively major changes made in the 1976 UBC, this code was used as a milestone. Primarily caused by life-threatening damage to various building types in the 1994 Northridge earthquake and subsequent code changes, a relatively complex set of milestone years was developed. ASCE 31 contains such a set of code milestone years for each of the several codes used in this country over the last thirty years. Although ASCE 31 suggests that compliance with these codes is only a recommended cut-off to not require evaluations for life safety, in all but very unusual situations, the table can be accepted. This table, Table 3-1 of ASCE 3,1 is reproduced as Figure 8-7.

● Initial Evaluation (ASCE 31 Tier 1)

The ASCE Tier 1 evaluation is similar to FEMA's Rapid Evaluation in that it is based on the model building type and certain characteristics of the building. The significant difference is that structural drawings, or data equivalent to structural drawings, are required to complete the evaluation, and the evaluation will take several days rather than several hours. After identifying the appropriate FEMA Building Type, a series of prescriptive requirements are investigated, most of which do not require calculations.

If the building is found to be noncompliant with any requirement, it is potentially seismically deficient. After completing the investigation of a rather exhaustive set of requirements, the engineer reviews the list of requirements with which the building does not comply, and decides if the building should be categorized as noncompliant or deficient. A conservative interpretation of the method is that any single noncompliance is sufficient to fail the building, but most engineers exercise their judgement in cases of noncompliance with only a few requirements. Historically, this method has developed with the pass/fail criterion of life safety, but the final ASCE Standard includes criteria for both Life Safety and Immediate Occupancy, a performance more closely related to continued use of the building. Because of the importance associated with the Immediate Occupancy performance level, a building cannot pass these requirements with only a Tier 1 analysis.

Table 3-1. Benchmark Buildings

Building Type[1, 2]	Model Building Seismic Design Provisions					FEMA 178[ls]	FEMA 310[ls, lo]	CBC[lo]
	NBC[ls]	SBC[ls]	UBC[ls]	IBC[ls]	NEHRP[ls]			
Wood Frame, Wood Shear Panels (Type W1 & W2)	1993	1994	1976	2000	1985	*	1998	1973
Wood Frame, Wood Shear Panels (Type W1A)	*	*	1997	2000	1997	*	1998	1973
Steel Moment Resisting Frame (Type S1 & S1A)	*	*	1994[4]	2000	**	*	1998	1995
Steel Braced Frame (Type S2 & S2A)	1993	1994	1988	2000	1991	1992	1998	1973
Light Metal Frame (Type S3)	*	*	*	2000	*	1992	1998	1973
Steel Frame w/ Concrete Shear Walls (Type S4)	1993	1994	1976	2000	1985	1992	1998	1973
Reinforced Concrete Moment Resisting Frame (Type C1)[3]	1993	1994	1976	2000	1985	*	1998	1973
Reinforced Concrete Shear Walls (Type C2 & C2A)	1993	1994	1976	2000	1985	*	1998	1973
Steel Frame with URM Infill (Type S5, S5A)	*	*	*	2000	*	*	1998	*
Concrete Frame with URM Infill (Type C3 & C3A)	*	*	*	2000	*	*	1998	*
Tilt-up Concrete (Type PC1 & PC1A)	*	*	1997	2000	*	*	1998	*
Precast Concrete Frame (Type PC2 & PC2A)	*	*	*	2000	*	1992	1998	1973
Reinforced Masonry (Type RM1)	*	*	1997	2000	*	*	1998	*
Reinforced Masonry (Type RM2)	1993	1994	1976	2000	1985	*	1998	*
Unreinforced Masonry (Type URM)[5]	*	*	1991[6]	2000	*	1992	*	*
Unreinforced Masonry (Type URMA)	*	*	*	2000	*	*	1998	*

1 Building Type refers to one of the Common Building Types defined in Table 2-2.
2 Buildings on hillside sites shall not be considered Benchmark Buildings.
3 Flat Slab Buildings shall not be considered Benchmark Buildings.
4 Steel Moment-Resisting Frames shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.
5 URM buildings evaluated using the ABK Methodology (ABK, 1984) may be considered benchmark buildings.
6 Refers to the GSREB or its predecessor, the UCBC (Uniform Code of Building Conservation).

ls Only buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Life-Safety Performance Level may be considered Benchmark Buildings.
lo Buildings designed and constructed or evaluated in accordance with these documents and being evaluated to either the Life-Safety or Immediate Occupancy Performance Level may be considered Benchmark Buildings.

* No benchmark year; buildings shall be evaluated using this standard.
** Local provisions shall be compared with the UBC.

NBC—Building Officials and Code Administrators, *National Building Code.*
SBC—Southern Building Code Congress, *Standard Building Code.*
UBC—International Conference of Building Officials, *Uniform Building Code.*
GSREB—International Conference of Building Officials, *Guidelines for Seismic Retrofit of Existing Buildings.*
IBC—International Code Council, *International Building Code.*
NEHRP—Federal Emergency Management Agency, *NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings.*
CBC—California Building Standards Commission, *California Building Code, California Code of Regulations, Title 24.*

Figure 8-7: From ASCE 31-03, showing building types that might be considered “safe” due to the code under which they were designed.

SOURCE: ASCE 31-01, *SEISMIC EVALUATION OF EXISTING BUILDINGS*, THE STRUCTURAL ENGINEERING INSTITUTE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS, 2003 (REPRINTED WITH PERMISSION OF ASCE).

Intermediate Evaluation (ASCE 31 Tier 2)

The ASCE intermediate level of evaluation, called Tier 2, is similar in level of effort of historical nonstandardized methods. Normally, an analysis of the whole building is performed and the equivalents of stress checks are made on important lateral force-resisting components. This analysis is done in the context and organization of the set of requirements used in Tier 1, but the process is not unlike seismic analysis traditionally performed for both evaluation and design of new buildings. ASCE 31 includes the requirements for both the LifeSafety and Immediate Occupancy performance levels for a Tier 2 Evaluation.

Detailed Evaluation (ASCE 31 Tier 3)

The most detailed evaluations are somewhat undefined because there is no ceiling on sophistication or level of effort. The most common method used in Tier 3 is a performance evaluation using FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings* (FEMA, 2000), based on simplified nonlinear analysis using pushover analysis (see Chapter 6). This method approximates the maximum lateral deformation that the building will suffer in a design event, considering the nonlinear behavior created by yielding and damage to components. The level of deformation of individual components is compared with standard deformations preset to performance levels of Collapse Prevention, Life Safety, and Immediate Occupancy, with the probable damage state of the building as a whole set at that level of the worst component. Efforts are being made to more realistically relate the damage states of all the components to a global damage state. This method can be used either to determine the probable damage state of the building for evaluation purposes, or to check the ability of a retrofit scheme to meet a target level.

With the advancement of computer capability and analysis software, nonlinear analysis techniques are constantly being improved. The ultimate goal, although not expected to be an everyday tool in the near future, is to simulate the movements of the full buildings during an entire earthquake, including the constantly changing properties of the structural components due to yielding and damage. The overall damage to various components is then accumulated and the global damage state thereby surmised.

8.4.3 Other Evaluation Issues

There are several other issues associated with seismic evaluation that should be recognized. Only three will be discussed here. First is the data required to perform competent evaluations at the various levels, as discussed above. Second, it is important to understand the performance expectation of the pass-fail line for various evaluation methodologies. Last, the reliability (or lack thereof) of the methods and of evaluation and/or performance prediction in general,should be recognized.

● Data Required for Seismic Evaluation

Obviously, for methods depending on the FEMA building type, the building type must be known. In fact, there are other similar classifications of building types also used to define building performance at the broadest level. Using data as discussed in paragraph 8.4.1, crude expectations of performance and therefore comparative evaluation can be completed. Most such systems, however, are refined by age, physical condition of the building, configuration, and other more detailed data, when available. Most "rapid" evaluation methods, based on building type and very basic building characteristics, do not require structural drawings. Responsible evaluators will insist on a site visit (in many cases to make sure the building is still there, if nothing else).

The more standardized evaluation methods discussed in paragraph 8.4.2 essentially require drawings. If detailed structural drawings are not available, simple evaluations of some model building types (wood buildings, tilt-ups, and sometimes URM) can be performed based on layout drawings or from data prepared from field visits. However, when reinforced concrete, reinforced masonry, or structural steel is a significant part of the structure, it is most often economically infeasible to reproduce "as-built" drawings. Practically in those cases, with rare exceptions, the building is deduced to be in nonconformance and, as a retrofit, a new seismic system is introduced to render the unknowns of the existing structure insignificant. Even in those cases, however, extensive field work is necessary to produce enough structural data to create a reasonable set of construction documents.

If original structural drawings are available that are confirmed to be reasonably accurate from spot checks in the field, most evaluation techniques can be employed. However, material properties are often not included on the drawings and must be deduced from the era of construction. Deterioration can also affect the material properties of several building types. Often the potential variability in the analysis due to different possible combinations of material properties requires in-situ testing of material properties. The techniques for this testing are well established, but cost and disruption to tenants are often an issue.

As explained in Chapter 3, “Site Evaluation and Selection”, many areas of the country are mapped in detail for seismic parameters related to design, although such parameters continue to be investigated and updated. When warranted, site-specific studies can be performed to obtain timely and locally derived data. However, other seismic site hazards, such as liquefaction, landslide, and potential surface fault rupture, are less well mapped and may require a site-specific study, if there is reason to suspect their potential at a site.

Perhaps a less obvious important characteristic of a site is the detail of adjacent structures. Particularly in urban settings, adjacent buildings often have inadequate separation or are even connected to the building to be evaluated. Although legal issues abound when trying to deal with this issue, it is unrealistic to analyze and evaluate such a building as if it were freestanding. Formal evaluation techniques, such as ASCE 31, have addressed this issue, at least for buildings that are not connected, by highlighting the conditions known to potentially produce significant damage. First, if floors do not align between adjacent buildings and pounding is expected, the stiff floor from one building could cause a bearing wall or column in the adjacent building to collapse. Secondly, if buildings are of significantly different height, the interaction from pounding has been observed to cause damage. See Figure 8-8.

● Performance Objectives and Acceptability

Traditionally, evaluation techniques have been targeted at determining if a building is adequately life safe in an earthquake, similar to code goals for new buildings. However, as discussed in section 8.2.2, a standard different and less than that used for new buildings evolved, but was still termed life safety. Only with the development of performance-based engineering did evaluation methods aimed at other performance standards emerge. Even life safety has proven to be amorphous over the years and often has been defined by the evaluation technique du jour. Seismically, life safety is a difficult concept, due to the huge potential variation in ground motion and the many sources of damage that could cause injury

Figure 8-8: Typical adjacency issues in urban settings.

The small building in the center clearly cannot fail side to side, but this condition is not considered in mandatory retrofit ordinances that assume the adjacent buildings may be removed. In fact, however, the small building is in great danger from falling debris from its taller neighbors. In addition, the taller buildings are at risk from receiving serious damage to the corner columns from pounding against the shorter building.

or death. However, the term is well embedded in public policy and continues to persist in seismic codes and standards.

FEMA, in sponsoring the development of FEMA 273 (and later FEMA 356), wanted a more specific definition of a suitable goal for seismic safety, and thus the **Basic Safety Objective** (BSO) was defined. This performance objective consists of two requirements: the building would provide life safety for the standardized code event and, in addition, the building would not collapse in the **Maximum Considered Event** (MCE), a very rare event now defined by code. Since these FEMA documents are non-mandatory (unless locally adopted), the BSO has not become a widely accepted standard (the BSO also includes mandatory nonstructural minimum requirements, which also may delay its wide acceptance).

Chapter 6 contains a detailed discussion of performance-based engineering, which is gaining acceptance for evaluations at any level. But performance characterization in various forms has been used for some time, primarily to set policy. Such policies require descriptions of various performance levels, even if the technical ability to define or predict the various levels often lagged behind. Table 8-2 shows several such performance descriptions, many developed decades ago, which have been used to set policy—most concentrating on life safety. The table is set up to approximately equilibrate levels of performance across horizontal lines.

The first columns in Table 8-2 describe a system used by the University of California. GOOD, the best performance, is defined as the equivalent life safety as that provided by the code for new buildings—but without consideration of monetary damage. The next level below was set at the acceptable level for evaluation, while retrofits are required to meet the GOOD level.

The next column, labeled "DSA", is a Roman numeral system developed by the California Division of the State Architect for use with state-owned buildings. Each level has a description of damage and potential results of damage ("building not reocccupied for months") but no reference to engineering parameters. The state used an acceptance level of IV, but set the goal for retrofits to III.

The levels described in the next columns come from one of the early developments of performance based earthquake engineering, *Vision 2000*, developed by the Structural Engineers Association of California. It is a relatively comprehensive scale using five primary descriptions of damage, each with a "plus" and "minus", resulting in ten levels.

Finally, to indicate a perhaps more commonly recognized standard of performance, are the three occupancy tagging levels of Red, Yellow, and Green used for emergency evaluation immediately after damaging earthquakes.

Reliability of Seismic Evaluations

The most significant characteristic in the design of buildings for earthquakes is the variability of ground motions. Not only do magnitudes and locations vary, but also the effects of fault rupture, wave path, and local

Table 8-2: A comparison of performance classifications

COMPARATIVE CROSS REFERENCE OF VARIOUS SEISMIC PERFORMANCE RATINGS							
UC Rating			DSA* Rating	SEAOC Vision 2000 Rating			ATC-20 Post-Earthquake
Designation	Structural Damage Level	Hazard to Life		Numerical Rating	Performance Expectation	Anticipated Damage	Assessment Designation
GOOD	Some Damage	Not Significant	I	10	Fully Operation	Negligible	"Green Tag" Inspected No Restriction on Use or Occupancy
			II	9			
			II	8	Operational	Light	
			III	7			
			III	6	Life Safety	Moderate	
			IV	5			"Yellow Tag" Limited Entry Off Limits to Unauthorized Personnel
FAIR	Damage	Low	IV	4	Near Collapse	Severe	
POOR	Significant Damage	Appreciable	V	3			
			VI	2	Partial Collapse	Complete	"Red Tag" Unsafe Do Not Enter or Occupy
VERY POOR	Extensive Damage	High	VI	2	Partial Collapse (Large Assembly Area Only)		
			VII	1	Total Collapse		

site soils create a literally infinite set of possible time histories of motion. Studies have shown that time histories within a common family of parameters used for design (response spectrum) can produce significantly different responses. This variation normally dominates over the scatter of results from analysis or evaluation techniques.

However, codes for new buildings can require many limitations of material, lateral system, configuration, and height that will reasonably assure acceptable performance, particularly the prevention of collapse. These same limitations can seldom be applied to existing buildings, so the variation of actual performance is expected to be much larger. In addition, the cost of retrofit is often high, and attempts have been made to avoid unnecessary conservatism in evaluation methodologies. It is probable, therefore, that a significant number of buildings may fail to perform as evaluated, perhaps in the range of 10% or more. No comprehensive study has been made to determine this reliability, but ongoing programs to further develop performance-based seismic engineering are expected to estimate the variability of evaluation results and refine the methods accordingly.

Box 2 Describing Seismic Performance

Seismic performance is specified by selecting a maximum tolerable damage level for a given earthquake-shaking intensity. The shaking intensity can be specified **probabilistically**, derived by considering all future potential shaking at the site regardless of the causative fault, or **deterministically**, giving the expected shaking at the site for a given-sized earthquake on a given fault. The damage level can be described using one of several existing scales, including the DSA Risk Levels or performance levels developed in the long-running FEMA program to mitigate seismic risks from existing buildings.

Describing Shaking Intensity

For some time, the earthquake shaking used by the building code for new buildings has been described probabilistically, as shaking with a 10% chance of being exceeded in a 50-year time period (50 years being judged as the average life of buildings). This can also be specified, similar to methods used with storms or floods, as the shaking with a return period of 475 years. (Actually, for ease of use, the return period is often rounded to 500 years, and since actual earthquake events are more understandable than probabilistic shaking, the most common term, although slightly inaccurate, is “the 500-year event.”)

Nationally applicable building codes were therefore based on the level of shaking intensity expected at any site once every 500 years (on average). However, engineers in several areas of the country, most notably Salt Lake City, Utah; Charleston, South Carolina; and Memphis, Tennessee, felt that this standard was not providing sufficient safety in their regions because very rare, exceptionally large earthquakes could occur in those areas, producing shaking intensities several times that of the 500 year event. Should such a rare earthquake occur, the building code design would not provide the same level of protection provided in areas of high seismicity, particularly California, because rare, exceptionally large shaking in California is estimated to be only marginally larger (about 1.5 times) than the 500-year shaking. It was therefore decided to determine the national mapping parameters on a much longer return period—one that would capture the rare events in the regions at issue, and a 2,500 year event was chosen (known as the Maximum Considered Event—MCE). Finally, it was judged unnecessary, and in fact undesirable, to significantly change seismic design practices in California, so the MCE was multiplied by 2/3 to make California design shaking levels about the same as before.

continued next page

(If the new shaking level - about 1.5 times the old - were multiplied by 2/3, the final design parameter would not change.) However, in a region where the MCE is 3 times the previously used 500-year event, the new parameter of 2/3 MCE would result in a shaking level twice that previously used—providing the sought-after additional level of safety in those regions. Currently, national standards such as ASCE 31 define the level of shaking to be considered for evaluation of existing buildings to be 2/3 MCE, which, as previously explained, is about the same as the 500-year event for much of California.

Describing Damage Levels

Although several descriptions of performance damage levels are currently in use (see Table 8-2), descriptions of FEMA performance levels summarized from FEMA 356 (FEMA, 2000), which covers the full range of performance, are given below:

- **Operational**: Buildings meeting this performance level are expected to sustain minimal or no damage to their structural and nonstructural components. The building will be suitable for its normal occupancy and use, although possibly in a slightly impaired mode, with power, water, and other required utilities provided from emergency sources. The risk to life safety is extremely low.
- **Immediate Occupancy**: Buildings meeting this performance level are expected to sustain minimal or no damage to their structural elements and only minor damage to their nonstructural components. Although immediate re-occupancy of the building will be possible, it may be necessary to perform some cleanup and repair and await the restoration of utility service to function in a normal mode. The risk to life safety is very low.
- **Life Safety**: Buildings meeting this performance level may experience extensive damage to structural and nonstructural components. Structural repair may be required before re-occupancy, and the combination of structural and nonstructural repairs may be deemed economically impractical. The risk to life safety is low.
- **Collapse Prevention**: Buildings meeting this performance level will not suffer complete or partial collapse nor drop massive portions of their structural or cladding on to the adjacent property. Internal damage may be severe, including local structural and nonstructural damage that poses risk to life safety. However, because the building itself does not collapse, gross loss of life is avoided. Many buildings in this damage state will be a complete economic loss.

8.5 SEISMIC REHABILITATION OF EXISTING BUILDINGS

There are many reasons why buildings might be seismically retrofitted, including renovations that trigger a mandatory upgrade, a building subjected to a retroactive ordinance, or the owner simply wanting (or needing) improved performance. The reason for the upgrade may influence the technique and thoroughness of the work, because owners faced with mandatory upgrade may seek out the least expensive, but approvable, solution, whereas an owner needing better performance will more likely be willing to invest more for a solution that addresses their particular concerns. There are many other factors that shape a retrofit solution, such as the type of deficiency present, if the building is occupied, and the future use and aesthetic character of the building.

The continuing improvement of analysis techniques and the emergence of performance-based design are also having a large effect on retrofit schemes, by enabling engineers to refine their designs to address the specific deficiencies at the desired performance level. In many cases, however, the retrofits are becoming controlled by the brittleness of existing components that must be protected from excess deformation with systems that may be stronger and/or stiffer than those used for new buildings. Some older retrofits, done to prescriptive standards or using now-outdated strengthening elements borrowed from new building designs, may themselves be deficient, depending on the desired performance. Seismic retrofit analysis, techniques, and components, similar to new building technology, are not static, and applications should be regularly reviewed for continued effectiveness.

8.5.1 Categories of Rehabilitation Activity

In most cases, the primary focus for determining a viable retrofit scheme is on vertically oriented components (e.g. column, walls, braces, etc.) because of their significance in providing either lateral stability or gravity-load resistance. Deficiencies in vertical elements are caused by excessive inter-story deformations that either create unacceptable force or deformation demands. However, depending on the building type, the walls and columns may be adequate for seismic and gravity loads, but the building is inadequately tied together, still forming a threat for partial or complete collapse in an earthquake. It is imperative to have a thorough understanding of the expected seismic response of the existing building, and all of its deficiencies to design an efficient retrofit scheme. There are three basic categories of measures taken to retrofit a building:

1) Modification of global behavior, usually decreasing deformations (drifts);

2) Modification of local behavior, usually increasing deformation capacity;

3) Connectivity, consisting of assuring that individual elements do not become detached and fall, assuring a complete load path, and assuring that the force distributions assumed by the designer can occur.

The types of retrofit measures often balance one another, in that employing more of one will mean less of another is needed. It is obvious that providing added global stiffness will require less deformation capacity for local elements (e.g. individual columns), but it is often less obvious that careful placement of new lateral elements may minimize a connectivity issue such as a diaphragm deficiency. Important connectivity issues such as wall-to-floor ties, however, are often independent and must be adequately supplied.

● Modification of Global Behavior

Modification to global behavior normally focuses on deformation, although when designing to prescriptive standards, this may take the form of adding strength. Overall seismic deformation demand can be reduced by adding stiffness in the form of shear walls or braced frames. Addition of moment frames is normally ineffective in adding stiffness. New elements may be added or created from a composite of new and old components. Examples of such composites include filling in openings of walls and using existing columns for chord members for new shear walls or braced frames.

Particular ground motions have a very specific deformation demand on structures with various periods, as discussed in Chapter 4. Given an equal period of vibration, this deformation will occur, whether distributed over the height of the building or concentrated at one floor. If one or more inter-story drifts are unacceptable, it may be possible to redistribute stiffness vertically to obtain a more even distribution of drift. A soft or weak story is an extreme example of such a problem. Such stories are usually eliminated by adding strength and stiffness in such a way as to more closely balance the stiffness of each level, and thus evenly spread the deformation demand over the height of the structure.

Seismic isolation is the supreme example of the concept of redistribution of deformation. Essentially all deformation is shifted to bearings, placed at the isolation level, that are specifically designed for such response. The bearings limit the response of the superstructure, which can be designed to remain essentially undamaged for this maximum load. The feasibility of providing isolation bearings that limit superstructure accelerations to low levels not only facilitates design of superstructures to remain nearly elastic, but also provides a controlled environment for design of nonstructural systems and contents.

Global deformations can also be controlled by the addition of passive energy dissipation devices, or **dampers**, to the structure. Although effective at controlling deformations, large local forces may be generated at the dampers that must be transferred from the device to structure and foundation, and the disruptive effect of these elements on the interior of the building is no different than a rigid brace.

● Modification of Local Behavior

Rather than providing retrofit measures that affect the entire structure, deficiencies also can be eliminated at the local component level. This can be done by enhancing the existing shear or moment strength of an element, or simply by altering the element in a way that allows additional deformation without compromising vertical-load carrying capacity.

Given that in most cases, that certain components of the structure will yield (i.e., become inelastic), some yielding sequences are almost always benign: beams yielding before columns, bracing members yielding before connections, and bending yielding before shear failure in columns and walls. These relationships can be determined by analysis and controlled by local retrofit in a variety of ways. Columns in frames and connections in braces can be strengthened, and the shear capacity of columns and walls can be enhanced to be stronger than the shear that can be delivered.

Concrete columns can be wrapped with steel, concrete, or other materials to provide confinement and shear strength. Concrete and masonry walls can be layered with reinforced concrete, plate steel, and other materials. Composites of glass or carbon fibers and epoxy are becoming popular to enhance shear strength and confinement in columns, and to provide strengthening to walls.

Another method to protect against the collapse risk posed by excess drift is to provide a supplementary gravity support system for elements that might be unreliable at expected high-deformation levels. For example, supplementary support for concentrated wall-supported loads is a requirement in California standards for retrofit of unreinforced masonry buildings. In several cases, supplementary support has also been used in concrete buildings.

Lastly, deformation capacity can be enhanced locally by uncoupling brittle elements from the deforming structure, or by removing them completely. Examples of this procedure include placement of vertical saw cuts in unreinforced masonry walls to change their behavior from shear failure to a more acceptable rocking mode, and to create slots between spandrel beams and columns to prevent the column from acting as a "short column" prone to shear failure.

- Connectivity

Connectivity deficiencies are within the load path: wall out-of-plane connection to diaphragms; connection of diaphragm to vertical lateral force-resisting elements; connection of vertical elements to foundation; connection of foundation to soil. A complete load path of some minimum strength is always required, so connectivity deficiencies are usually a matter of degree. A building with a complete but relatively weak or brittle load path might be a candidate for retrofit by seismic isolation to simply keep the load below the brittle range.

The only location in the connectivity load path at which yielding is generally allowed is the foundation/structure interface. Allowing no movement at this location is expensive and often counterproductive, as fixed foundations transfer larger seismic demands to the superstructure. Most recently developed retrofit guidelines are attempting to provide simplified guidance to the designer on how to deal with this difficult issue and minimize foundation costs.

8.5.2 Conceptual Design of a Retrofit Scheme for an Individual Building

There are many specific methods of intervention available to retrofit designers, as previously discussed. The selection of the specific type of element or system is dependent on local cost, availability, and suitability for the structure in question. Any system used to resist lateral load in new buildings can also be used for retrofit. It is thus an extensive task to develop guidelines for such selection. In addition, as in the design of a new building, there is usually a choice of where to locate elements, although it is generally more restrictive in existing buildings. However, in the end, there are nonseismic issues associated with each building or project that most often control the specific scheme to be used.

The solution chosen for retrofit is almost always dictated by building user-oriented issues rather than by merely satisfying technical demands. There are five basic issues that are always of concern to building owners or users: **seismic performance, construction cost, disruption to the building users during construction** (often translating to a cost), **long-term affect on building space planning**, and **aesthetics**, including consideration of historic preservation.

All of these characteristics are always considered, but an importance will eventually be put on each of them, either consciously or subconsciously, and these weighting factors invariably will determine the scheme chosen.

❍ Seismic performance

Prior to the emphasis on performance-based design, perceived qualitative differences between the probable performance of difference schemes were used to assist in choosing a scheme. Now, specific performance objectives are often set prior to beginning development of schemes. Objectives that require a very limited amount of damage or “continued occupancy” will severely limit the retrofit methods that can be used and may control the other four issues.

❍ Construction cost

Construction cost is always important and is balanced against one or more other considerations deemed significant. However, sometimes other economic considerations, such as the cost of disruption to building users, or the value of contents to be seismically protected, can be orders of magnitude larger than construction costs, thus lessening its importance.

❍ Disruption to the building users during construction

Retrofits are often done at the time of major building remodels, and this issue is minimized. However, in cases where the building is partially or completely occupied, this parameter commonly becomes dominant and controls the design.

- ❍ Long-term effect on building space planning

  This characteristic is often judged less important that the other four and is therefore usually sacrificed to satisfy other goals. In many cases, the planning flexibility is only subtly changed. However, it can be significant in building occupancies that need open spaces, such as retail spaces and parking garages.

- ❍ Aesthetics

  In historic buildings, considerations of preservation of historic fabric usually control the design. In many cases, even performance objectives are controlled by guidelines imposed by preservation. In non historic buildings, aesthetics is commonly stated as a criterion, but in the end is often sacrificed, particularly in favor of minimizing cost and disruption to tenants.

These parameters can merely be recognized as significant influences on the retrofit scheme or can be used formally to compare schemes. For example, a comparison matrix can be developed by scoring alternative schemes in each category and then applying a weighting factor deduced from the owner's needs to each category.

Figure 8-9 describes the evolution of a retrofit scheme based on several changes in the owner's weighting of these five characteristics.

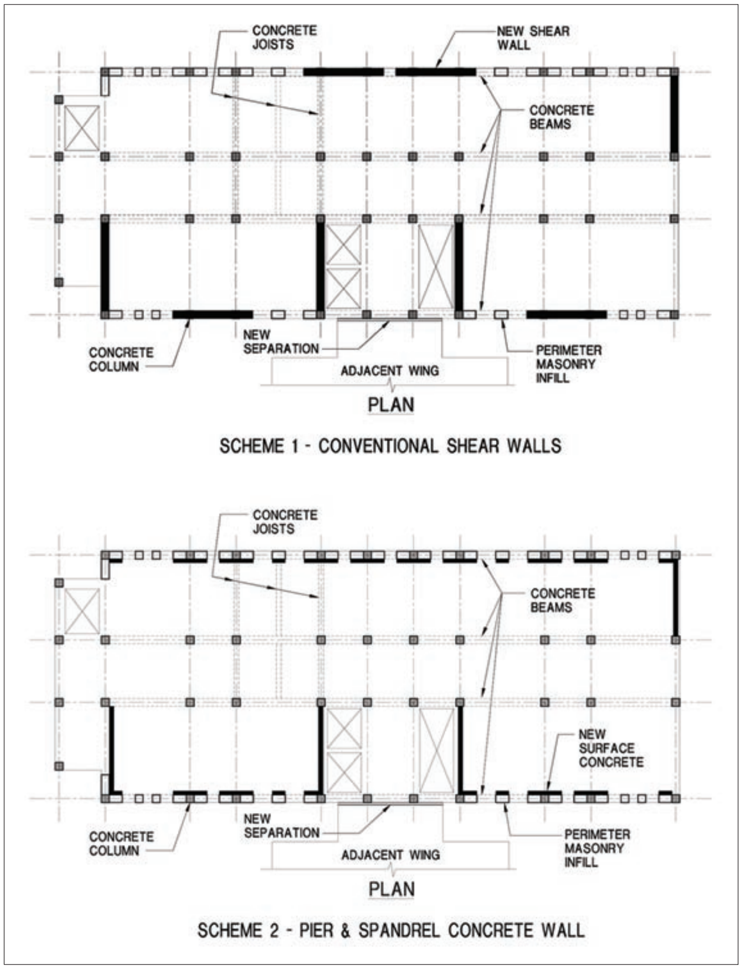
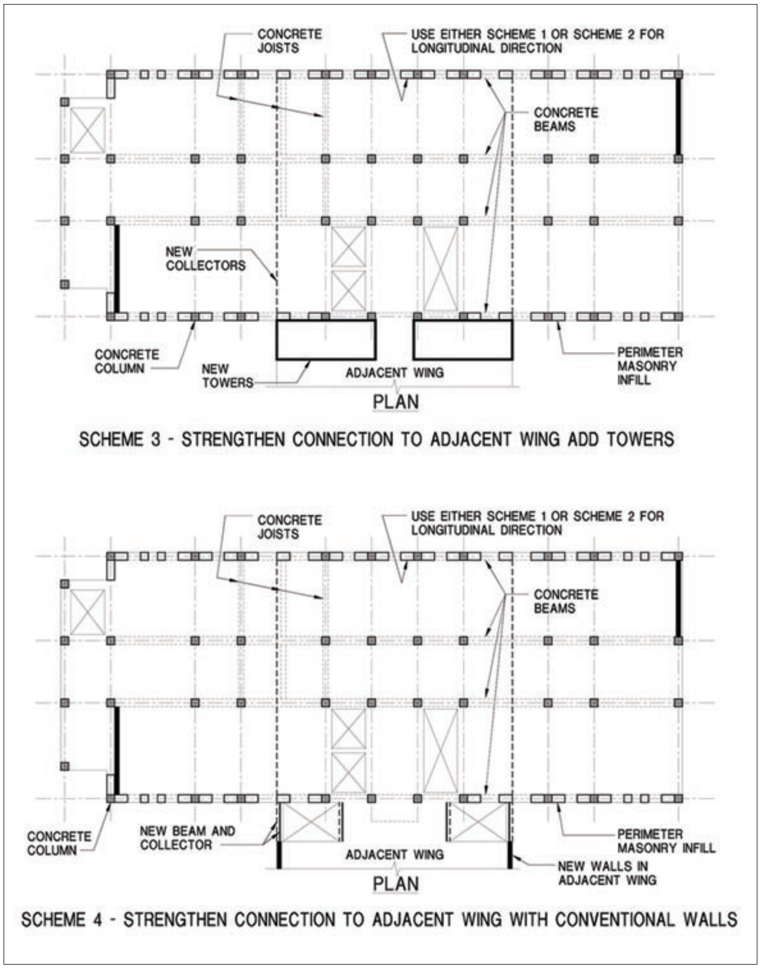


Figure 8-9a: Example of effect of non technical issues on retrofit schemes.

This is a seven-story concrete building built in the early 1920s. It consists of two wings in a T shape. The plans show the second of the two wings poorly connected at the location indicated. The building is a concrete frame with brick infill exterior walls, and lateral forces were not apparently considered in the original design. Although not officially judged historic, the exterior was articulated and considered pleasing and a good representative of its construction era. As can be seen in the plan, the building has no lateral strength in the transverse direction other than the poor connection to the second wing, and was evaluated to present a high risk to occupants.

It was judged early that the vertical load carrying elements had little drift capacity, and that stiffening with shear walls was the only feasible solution. The first two schemes shown are straightforward applications of shear walls. The first concentrates the work to minimize disruption, but closes windows and creates large overturning moments at the base. The second distributes the longitudinal elements and preserves windows by using a pier-spandrel shear wall. Both schemes separated the wings into two buildings, to allow future demolition of either to facilitate phasing for a new replacement building sometime in the future. The cost and disruption was judged high, and the work would have to be phased upwards, evacuating three floors at a time to avoid the noise and disruption.

Figure 8-9b: Example of effect of non technical issues on retrofit schemes.



Schemes 1 and 2 required a complete lateral system in both buildings because of the separation installed at the wing intersection, which also caused difficult exiting issues. Schemes 3 and 4 were therefore developed, providing a strong inter-tie between wings and taking advantage of several new lateral elements to provide support to both wings. Scheme 3 featured new concrete towers as shown. Although the outside location was considered advantageous from a disruption standpoint, the towers closed windows and caused disruption to mechanical services. Scheme 4 was similar to 3, but eliminated the towers. Schemes 3 and 4 had less construction cost than 1 and 2, but disruption, in terms of phasing, caused essentially the same total downtime.

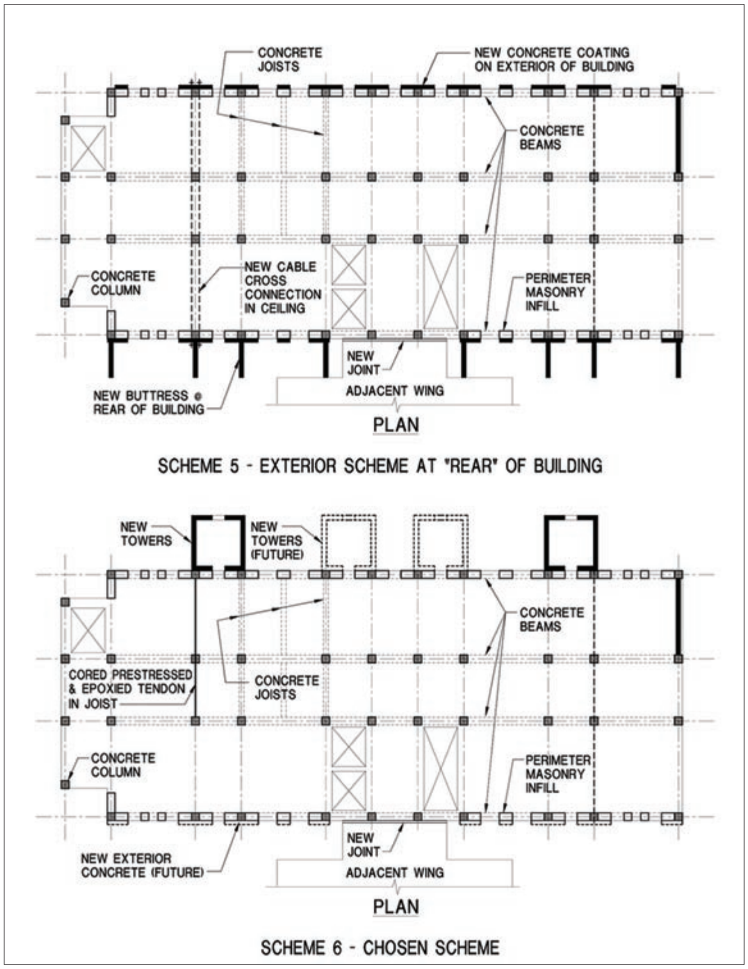


Figure 8-9c: Example of effect of non technical issues on retrofit schemes.

When the owner completed a study of the availability and cost of surge space in the area to facilitate the phasing required for Schemes 1–4, it was discovered that the cost of moving and rental space was larger by far than the construction costs thus far budgeted.

Occupant disruption thus became the primary control parameter for development of retrofit schemes, and aesthetics, measured by preservation of the exterior appearance, was significantly reduced as a consideration. Scheme 5 was developed with buttresses on the off-street side of the building and longitudinal walls applied from the outside. Collectors to the buttresses were to be post-tensioned cables installed through conduit placed at night in the ceiling spaces. Access to the rear of the building was difficult, so aesthetic considerations were further relaxed to allow buttresses—that became towers—on the front side. Collectors to the towers were post-tensioned rods installed in cores drilled approximately 16 feet (5 m) into the building in the center of 12-inch by 16-inch (30x40cm) joists. Both Schemes 5 and 6 installed a seismic separation at the intersection of wings to facilitate partial replacement. At the request of the contractor, another scheme, 6a, was developed that replaced the concrete shear walls with steel-braced frames, but it proved no more economical. Scheme 6 was selected for construction, although – since replacement at that time appeared to be in the long range planning stages – the owner chose to construct only part of Scheme 6, aimed at eliminating the obvious collapse mechanism in the transverse direction. As shown, only two towers were constructed, and the only longitudinal strengthening provided was the weak-way strength of the towers.

Figure 8-10: Intense construction activity and disruption from interior shotcrete.

SOURCE: RUTHERFORD & CHEKENE, CONSULTING ENGINEERS

Figure 8-11: Retrofit activities inside buildings are most often not surgical or delicate. Here work on a new foundation for a shear wall is prepared for casting.

SOURCE: RUTHERFORD & CHEKENE, CONSULTING ENGINEERS

8.5.3 Other Rehabilitation Issues

- Inadequate recognition of disruption to occupants

It is unfortunately common for the extent of interior construction and disruption to be underestimated. In many cases, occupants who were originally scheduled to remain in place are temporarily moved—at a significant increase in cost of the project—or the work is required to be done in off-hours, also a premium cost. Figures 8-10 and 8-11 indicate the level of construction intensity often required in retrofit.

Similarly, "exterior solutions," where strengthening elements are placed on the outside of the building are often more disruptive and noisier than

Figure 8-12: External towers were added to strengthen this building from the outside. The new tower is the element at center-left of the figure. The retrofit scheme for this building is discussed in detail in Figure 8-9.

anticipated and often require collector members to be placed on each floor within the building. Figure 8-12 shows the result of an exterior retrofit of adding towers on the outside of a building that, in fact, did not cause a single lost day of occupancy. High-strength steel rods were epoxied into horizontal cores, drilled twenty feet into the existing concrete beams to form the needed collectors.

- Collateral required work

As previously mentioned, retrofit work is often performed in conjunction with other remodeling or upgrading activities in a building. Such work normally triggers other mandatory improvements to the building, such as ADA compliance or life safety updating—all of which add cost to the project. However, even when seismic retrofit is undertaken by itself, the costs of ADA compliance, removal of disturbed hazardous material, and possibly life safety upgrades must be considered.

8.5.4 Examples

It is impossible to include examples that show the full range of structural elements and configurations used in seismic retrofit. There are definitely patterns, usually driven by economics or avoidance of disruption to occupants, but depending on the particular mix of owner requirements, as discussed in Section 8.5.2, thoughtful architects and engineers will always come up with a new solution.

Figure 8-13: Examples of the many configurations of steel braced frames used in commercial retrofit in San Francisco.

The retrofits were probably required by the URM Ordinance or triggered by upgrading or remodeling. Tall narrow brace configurations, as shown in the upper left and lower right, are less efficient that more flat brace orientations.

Figure 8-14: Examples of steel moment frames in similar commercial retrofits. Right: Note the large white column and double beam arrangement. Left: The moment frame is placed against the wall of the recess. The first floor columns are gray and the balance is pink. The frame can be seen on both the first and second floors.

Figure 8-15: Renovation and retrofit of a concrete warehouse structure by removing the exterior wall, inserting steel braces and window wall, and adding several floors.

Figure 8-16: Steel braced frames on exterior of building to avoid construction on the inside of the building.

Figure 8-17: A modern exterior buttress used for seismic strengthening.

SOURCE: TAKANE ESHIMA, MURAKAMI/NELSON

Figure 8-18: Infill of certain panels of an exterior wall for strengthening. Originally on the alley wall, only the central stair tower was solid wall, and elsewhere the upper window panel pattern was typical to the street.

Figure 8-19: Examples of addition of a new wall on the exterior of the building.

Left: The end wall has a new layer of concrete that wraps around the side for a short distance. This solution is unusual in an urban setting because of property lines. Right: a large academic building with more extensive C-shaped end elements. This solution facilitated construction while the building was occupied, although there was considerable noise and disruption experienced by the occupants.

Box 3 CASE STUDY: SEISMIC RETROFIT BREMERTON NAVAL HOSPITAL WASHINGTON

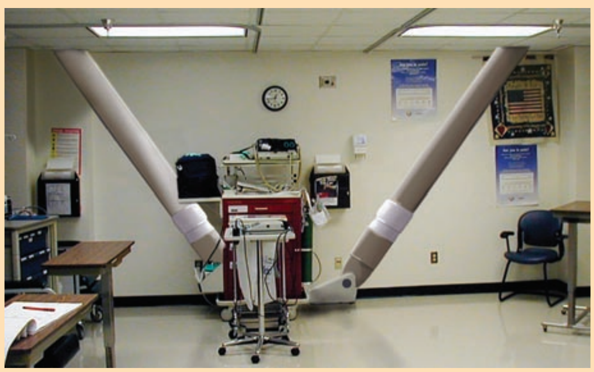
The U.S. Navy recognized in the late 1990s that the Bremerton Naval Hospital in Bremerton, Washington, was important not only for the 60,000 military families in the area, but also that it might be called upon to serve more than 250,000 people in the immediate area in the event of a major earthquake. Accordingly, a detailed seismic evaluation of the hospital using performance-based design engineering standards (FEMA 310, *Handbook for the Seismic Evaluation of Buildings*, A Prestandard and FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*) was performed to gain a better understanding of the potential seismic deficiencies.

The building's lateral-resisting system, constructed in the "pre-Northridge" 1960s, is comprised of steel moment-resistant frames at all beam-column connections. Although highly redundant, it is too flexible, resulting in excessive drift. The cladding panel connections were not designed to accommodate the expected drifts from a design-level earthquake, and presented a potential falling hazard. Additionally, there was incompatibility between the flexible structure and the rigid concrete stair tower.

BREMERTON NAVAL HOSPITAL continued

This detailed evaluation was completed in late 2001. In February 2002, the magnitude 6.8 Nisqually earthquake shook the Puget Sound region. Shaking at the hospital was modest, because the earthquake epicenter was located approximately 30 miles away. A seismograph at the hospital recorded a horizontal peak acceleration of 0.11g at the basement level and a peak roof acceleration of 0.47g. Calculated peak roof displacements from this modest earthquake were over 6 inches (a floor-to-floor drift ratio of 0.5%).

Because a traditional seismic retrofit that strengthened and stiffened the moment frames would have been costly and disruptive, alternate retrofit design methods were evaluated.



The use of supplemental passive damping devices proved to be the best approach to improve the seismic performance of the structure by reducing drift, while minimizing disruption during construction. Seismic forces, displacements, and floor accelerations would be substantially reduced by dissipation of the earthquake's energy through heat created in the damping devices. A total of 88 seismic dampers were installed at 44 select locations in the building.

Target performance levels were "Immediate Occupancy" for the 10%/50-year Design Basis Earthquake (DBE) and "Collapse Prevention" for the 2%/50-year Maximum Considered Earthquake (MCE).

CREDITS:CASE STUDY BASED ON THE ARTICLE STRONG MEDICINE, AUTHORS DOUGLAS WILSON, PE: RUSSELL KENT, PE; STEPHAN STANEK, PE AND DAVID SWANSON, PE,SE; IN *MODERN STEEL CONSTRUCTION*, AMERICAN INSTITUTE OF STEEL CONSTRUCTION, CHICAGO, IL, FEBRUARY 2005.

Photographs of retrofit buildings, although often interesting, seldom can tell the full story of the development of the scheme, and if the majority of retrofit elements are inside or hidden, tell almost nothing. Some photos are shown here, but are not intended to demonstrate the full range of buildings that have successfully undergone seismic retrofit or the full range of solutions to individual problems. In addition, due to limited space, only one or two points are made with each photo, rather than a full case study.

8.6 SPECIAL ISSUES WITH HISTORIC BUILDINGS

Seismic evaluation and retrofit of historic buildings generate complex public policy issues for which few general rules can be identified. Restoration, or renovations of large and important historic buildings usually have considerable public and jurisdictional oversight, in addition to employing an experienced design team that includes a special historic preservation consultant. The control and oversight for less important buildings that have historic status at some level, or that may qualify for such status, are highly variable. Designers are cautioned to locally investigate approval procedures for alterations on such buildings as well as seismic requirement, for them.

8.6.1 Special Seismic Considerations

It has been recognized in most areas of high seismicity that local public policy concerning seismic retrofit triggers must include special considerations for historic buildings. As discussed in Section 8.2, initial seismic safety criteria for existing buildings were focused on requirements for new buildings, which were marginally appropriate for most older buildings, but completely inappropriate for historic buildings. Special allowances were therefore created for archaic materials that were not allowed in new buildings, and the overall seismic upgrade level was lowered to reduce work that could compromise historic integrity and fabric. These kinds of technical criteria issues have been somewhat mitigated by the completion of FEMA 356 and the emergence of performance-based earthquake engineering, because consideration of archaic materials and fine-tuning of performance levels are now part of the normal lexicon.

8.6.2 Common Issues of Tradeoffs

Many buildings in this country that qualify for historic status are not exceptionally old and can be made commercially viable. The changes that are needed for successful adaptive reuse will often conflict with strict preservation guidelines, and compromises are needed in both directions to achieve a successful project that, in the end, could save the building from continuing decay and make it more accessible to the public. These tradeoffs occur in many areas of design, but seismic upgrading work often requires interventions that are not needed for any other reason. These interventions often fall under historic preservation guidelines that call for clear differentiation of new structural components, or that discourage recreation of historic components that are removed. As previously indicated, there are no rules for these conditions, and the most appropriate solution for each case must be determined individually.

Another common conflict is between current preservation of historic fabric and future preservation of the building due to the chosen seismic performance level. Typically, a better target performance in the future, possibly preventing unrecoverable damage, requires more seismic renovation work now. Most historic preservation codes allow lower expected seismic performance to reduce construction work and minimize damage. Like many seismic policies, there have not been enough earthquakes with seismically damaged historic buildings to test this general philosophy. In an ever-growing number of cases of important buildings, this dilemma has been addressed using seismic isolation—which by reducing loading to the superstructure, reduces required construction work and also reduces expected damage in future earthquakes. Typically, however, installing isolation into an existing building is expensive and may require a significant public subsidy to make viable. Several high-profile city hall buildings such as San Francisco, Oakland, Berkeley (after the 1989 Loma Prieta earthquake), and Los Angeles (after the 1994 Northridge earthquake) have been isolated, with FEMA assistance as part of post earthquake damage repairs.

8.6.3 Examples of Historical Buildings

The following illustrations show samples of seismic retrofit of historic buildings with brief descriptive notes. Complete discussion of the preservation issues and rehabilitation techniques of each case would be extensive and cannot be included here.

Figure 8-20: Several examples of historical buildings seismically retrofit and protected by seismic isolation.

Upper left: Oakland City Hall (Engineer: Forell/Elsesser). Upper right: San Francisco City Hall (Engineer: Forell/Elsesser). Bottom: Hearst Mining Building at University of California, Berkeley (Engineer: Rutherford & Chekene). Installation of an isolation system under an existing building is complex and often expensive, but the system minimizes the need to disrupt historic fabric in the superstructure with shear walls or braces, and is designed to protect the superstructure from significant damage in a major event.

Figure 8-21: Mills Hall, Mills College, California. Structure is wood studs and sheathing. Interior shear walls were created by installing plywood on selected interior surfaces.

SOURCE: RUTHERFORD & CHEKENE, CONSULTING ENGINEERS

Figure 8-22: St. Dominic's Church in San Francisco was seismically retrofitted using exterior buttresses.

SOURCE: RUTHERFORD & CHEKENE, CONSULTING ENGINEERS

Figure 8-23: University Hall, University of California, Berkeley. The structure is unreinforced brick masonry bearing wall with wood floors and roof—a classic unreinforced masonry building (URM). Lateral resistance was added with interior concrete shear walls, improvements to the floor and roof diaphragms, and substantial ties from the exterior walls to the diaphragms.

SOURCE: RUTHERFORD & CHEKENE, CONSULTING ENGINEERS

Figure 8-24: Original Quad arcades, Stanford University. The Quad, approximately 850 feet by 950 feet in plan, is surrounded by a covered arcade. Modern seismic retrofits have taken place over a 40-year period, with evolving techniques. Final sections were completed by removal of the interior wythe of sandstone, installation of reinforced concrete core, and reinstallation of the interior blocks, which were reduced in thickness. Due to environmental decay of many of the sandstone columns, additional seismic resistance was obtained in many locations by installation of precast concrete replicas that formed a part of a continuous vertical reinforced concrete member.

SOURCE: RUTHERFORD & CHEKENE, CONSULTING ENGINEERS

Figure 8-25: Stanford University Quad Corner Buildings. The four very similar corner buildings were also seismically strengthened over a 40-year period. Substantially different techniques evolved due to growing recognition of the historic value of the entire Quad. The first corner was "gutted" in 1962 and an entire new structure built inside with two a,dded floors. The second corner was done in 1977 and was also gutted, but the original floor levels were maintained and interior finishes were similar to the original. The last two corners, repaired and retrofitted following the 1989 Loma Prieta earthquake, were strengthened with interior concrete shear walls, while the bulk of the interior construction was maintained, including the wood floors and heavy timber roofs.

SOURCE: RUTHERFORD & CHEKENE, CONSULTING ENGINEERS

Figure 8-26: The museum at San Gabriel Mission, southern California, a historic adobe structure damaged by earthquakes in 1987 and 1994.It was repaired and strengthened, including installation of a bond beam of steel in the attic, anchor bolts into the wall, and stitching of cracks back together on the interior walls.

ENGINEER: MEL GREENE AND ASSOC.

Box 4 CASE STUDY: THE SEISMIC RETROFIT OF THE SALT LAKE CITY AND COUNTY BUILDING

The Salt Lake City and County Building was completed in 1894. It is now the seat of government for Salt Lake City, Utah. The historic landmark also housed offices for Salt Lake County until the 1980s but still retains its original name. The building is an unreinforced masonry bearing-wall structure with a 250-foot-high central clock tower (Figure A).

Figure A: The Salt Lake City and County Building.

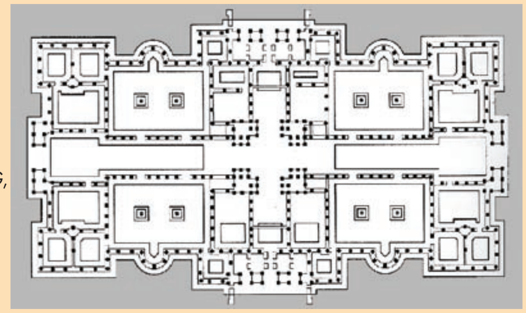
By the 1970s, it had become obvious that substantial repairs were needed if the building was to be of further service, and limited restoration was done between 1973 and 1979. However, the building remained a potential source of injuries and costly lawsuits, and in the 1980s much public controversy began regarding whether the building should be demolished or saved and restored. After many architectural end engineering studies, in 1986 the city council approved financing for the restoration of the building.

After extensive materials testing and structural analysis, the decision was made to use a base-isolation scheme, consisting of over 400 isolators that would be installed on top of the original strip footings, with a new concrete structural system built above the bearings to distribute loads to the isolators (Figure B).

Calculations showed a dramatic reduction in forced levels in the superstructure, so that shotcreting of existing walls would not be necessary, and consequently the historic interior finishes could be saved. Existing floor diaphragms would require only minimum strengthening around their perimeters. However, due to the significance of the towers as a seismic hazard, it was decided to use the results of conventional non-isolated analysis for sizing the steel space frame members used for the tower strengthening (Figure C).

Predicted damage from future earthquakes would also be substantially reduced, providing greatly increased safety to the building occupants. It would be necessary, however, to entirely

Figure B: Plan of isolator locations in basement.



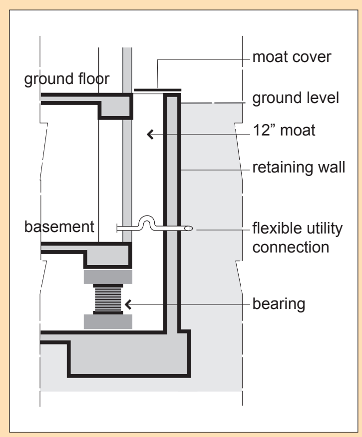
SOURCE: *SEISMIC ISOLATION RETROFITTING*, BY JAMES BAILEY AND EDMUND ALLEN, REPRINTED FROM THE VOLUME XX, 1988 ISSUE OF THE APT BULLETIN, THE JOURNAL OF THE ASSOCIATION FOR PRESERVATION TECHNOLOGY INTERNATIONAL

remove the first floor in order to provide space for the foundation work. In effect, the solution shifted the focus of the structural work from the shear walls above to the foundation, reducing much of the seismic retrofit work to a massive underpinning project.

Typical bearings used were approximately 17 inches square by 15 inches high and consisted of alternating layers of steel and rubber bonded together with a lead core (Figure D).

Figure C: Steel space frame inserted within tower

Figure D: Typical bearing layout with retaining wall and moat.



For the bearings to work properly, it was necessary to isolate the building from the ground horizontally. To do this, a retaining wall was constructed around the building perimeter with a 12-inch seismic gap (or moat), so that the building was free to move relative to the surrounding ground.

The retrofit of the Salt Lake City and County Building was completed in 1989 and was the world's first application of seismic base isolation for a historical structure.

CREDITS: PROJECT ARCHITECT: THE EHRENKRANTZ GROUP, NEW YORK. ASSOCIATE ARCHITECT: BURTCH BEALL JR. FAIA, SALT LAKE CITY, UT. STRUCTURAL ENGINEER: E.W. ALLEN AND ASSOCIATES, SALT LAKE CITY, UT. BASE ISOLATION CONSULTANT: FORELL/ELSESSER, SAN FRANCISCO, CA.

8.7 CONCLUSION

Table 8-3 summarizes common seismic deficiencies stemming from various site and configuration characteristics as well as those that might be expected in each FEMA model building type. See Section 8.2 for a discussion of "seismic deficiency" as used in this chapter and this table. Also included in Table 8-3 are retrofit measures that are often used for each situation.

8.8 REFERENCES

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Table 8-3: Common deficiencies by category of buildings

Category	Physical Characteristic	Performance Characteristics	Common Retrofit Techniques
Site	Liquefaction	• Small settlements from thin layers of liquefied material. • Large settlements and/or loss of foundation support from thick layers. • Horizontal flow possible if massive amounts of material liquefy, even with slight slopes.	• Stabilize soil with cement injection or by draining water to eliminate saturated state. • Place building on deep foundation that can withstand layer of liquefied material.
	Potential fault rupture	• If fault rupture is very near but not through building, only unique effect may be broken utilities or loss of access. • If fault rupture passes through building, severe damage to building is likely.	• Avoid condition if possible. • Retrofit possible in some cases with massive foundation that will force fault slippage around or under building without causing collapse of superstructure.
	Adjacent buildings	• If contact is expected, short adjacent buildings can cause a soft-story effect on the levels immediately above the short building. • If floors do not align, load-bearing columns or walls can be damaged by pounding, potentially causing collapse. • If walls share a structural wall ("common" wall), interaction may be extreme, and individual analysis is required. • Taller buildings, particularly URMs or buildings with URM exterior walls or parapets, may drop debris on shorter buildings, that potentially will pass through the roof.	• These conditions are difficult to mitigate without cooperation from both property owners. • Potential contact areas can be strengthened, but this may cause additional damage to neighboring building. • Supplemental vertical load system can be installed to prevent collapse caused by local damage. • The potential for falling debris from taller buildings can be minimized by adding supports and ties on adjacent building, and failure of the roof minimized with roof reinforcing.
Configuration	Soft/weak stories	• Disproportionate drift is concentrated on the soft story, potentially causing collapse. • A weak story may not be initially soft, but after yielding as a story first, it becomes soft, and displacements will concentrate in those elements already yielded, potentially causing collapse.	• The most straightforward retrofit is to add elements to the soft or weak story to force displacement from the earthquake to be more evenly distributed throughout the building height. • In some cases of soft stories, it is possible to soften other stories to be more evenly matched.
	Discontinuous wall/brace	• Shear walls or braces that do not continue to the foundation create forces at the level of discontinuity that must be designed for, including shear forces that must be transferred into the diaphragm and overturning tension and compression that must be resisted from below	• New walls or braces can be added below • The transfer forces can be made acceptable by reinforcement of the diaphragm and/or strengthening of columns below the ends of the wall/brace.
	Set back	• A setback often creates a dynamic discontinuity, because the story below is often much stiffer than the story above. This discontinuity can cause larger-than-expected demands on the floor immediately above.	• The floor above the setback can be strengthened to accept and smooth out the dynamic discontinuity.
	Plan Irregularity	• Plan irregularities such as L or T shapes often displace the center of mass from the center of lateral rigidity, causing torsion and resulting in high drifts on some elements. • Re-entrant corners often present in these buildings create large demands on floor diaphragms, tending to pull them apart at these negative corner conditions.	• Lateral force-resisting elements can be added to balance mass and resistance. • Chords and collectors can be added in diaphragms to resist re-entrant corner forces.

Table 8-3: Common deficiencies by category of buildings 2

Category	Physical Characteristic	Performance Characteristics	Common Retrofit Techniques
FEMA Model Building Types	W1: Small Wood Frame	• Masonry chimneys normally have incompatible stiffness with the structure and will fail themselves or pull away from the framing. • Cripple stud walls occurring only at the perimeter create a weak/soft story, that often causing the superstructure to topple over. • Discontinuities caused by large garage door openings cause damage. • Hillside structures with weak down-slope lateral anchorage or weak lateral force elements on the down-hill side fail and sometimes slide down the hill. • Buildings with lateral bracing of only stucco or gypsum board can suffer high economic damage.	• Masonry chimneys, reinforced or not, are difficult to make compatible with normally constructed houses. Factory-made light chimneys are the best option. • Most other deficiencies can be mitigated with added elements or added connections to tie the structure together. • Economic damage to gypsum board and hard floor finishes is difficult to control.
	W1A Large Wood Frame	• Normally these buildings are more regular than houses, but could suffer similar deficiencies. • The potential infamous deficiency for this building type is the soft/weak story created by ground floor parking—the so-called tuck-under building. Buildings with parking under a concrete first-floor level seldom have the soft story typical of all-wood buildings.	• Mitigate deficiencies similar to W1s. • Add lateral force-resisting elements of wood walls, steel brace frames or moment frames to eliminate the soft/weak story.
	W2 Large Wood Post/ Beam	• Often create plan irregularity due to plan shape or weakness at lines of the storefronts.	• Add lateral force-resisting elements as required.
	S1 Steel Moment Frame	• "Pre-Northridge" welded frames may fracture at the beam-column joint. • Older riveted or bolted frames may also suffer damage at connections • These structures are often very flexible and could collapse from side sway ("P-Delta effect"). • Excessive drift can cause damage to other elements such as interior partitions, stairwells, or cladding.	• Joints can be strengthened. • A diagonal steel bracing or other new lateral force-resisting system can be added. • Dampers can be added to reduce drifts.
	S2 Steel Braced Frame	• Braces that are stronger than their connections may fail in the connection, completely losing strength. • Braces can buckle and lose stiffness, causing a significant change in the overall dynamic response. • Certain tube bracing with "thin walls' may fracture and completely lose strength.	• Connections can be strengthened. • Braces can be added or strengthened to lesson effects of buckling. • Thin-walled tubes can be filled with grout or otherwise strengthened to eliminate local wall buckling. • All buildings with a relatively stiff and complete lateral force-resisting system, brittle or not, are candidates for seismic isolation.
	S3 Steel Light Frame	• Tension-only braces yield and lengthen, becoming loose. • Braces are often removed.	• Buildings are very light, and deficiencies have seldom caused significant damage. However, braces can be added to add overall strength to building.

Table 8-3: Common deficiencies by category of buildings 3

Category	Physical Characteristic	Performance Characteristics	Common Retrofit Techniques
FEMA Model Building Types	S4 Steel w/ Concrete Shear Wall	• This is generally considered a reasonably good building because the shear walls will absorb energy, and the frame will provide reliable gravity load support. • In older buildings, the shear walls may be eccentrically located, causing torsion. • Buildings with exterior concrete pier-and-spandrel enveloping the steel frame may be governed by the concrete response, which may seriously degrade.	• Retrofit measures must be tailored to the specific building and its deficiencies. • Walls can be added to eliminate torsion. • New walls can be added to reduce overall demand on the concrete. • Existing concrete elements can be reinforced to be more ductile. • All buildings with a relatively stiff and complete lateral force-resisting system, brittle or not, are candidates for seismic isolation.
	S5 Steel w/Infill Masonry	• This building type is well known for its good performance in the 1906 San Francisco earthquake. There is considerable documentation that shows good performance before the fire. • The brick and steel in this frame are thought to act together to perform well. Cracking and deterioration of the brick, however, may require costly repairs. • Soft stories created by storefronts and entrances may be severely damaged.	• While in a standard evaluation, these buildings may fail the standard, but advanced analysis may show that the composite system requires only minor retrofit. • Shear walls can be added to work with the exterior walls • The walls can be shotcreted from the inside. • Braced steel frames can be added, but the post-buckling response must be considered. • All buildings with a relatively stiff and complete lateral force resisting system, brittle or not, are candidates for seismic isolation.
	C1 Concrete Moment Frames	• If designed in accordance with ductile concrete principals (early to mid 1970s, depending on location, this building type will perform well. • If designed and detailed without the special requirements for ductility, this building type could pose serious risk of collapse due to shear failure in the joints or columns and subsequent degradation of strength and stiffness.	• In regions of high seismicity, these buildings are difficult to retrofit by locally improving elements. More likely, a new lateral force system of concrete shear walls or steel braced frames will be required. Added damping will also be effective. • In regions of moderate or low seismicity, local confinement of columns and joint regions might be adequate.
	C2 Concrete Shear Walls Type 1	• A building in which gravity load is carried by the same walls that resist seismic loads is normally considered a higher risk than one with a vertical load-carrying frame (Type 2). However, the bearing-wall building usually has many walls and therefore a low level of demand on the walls. The performance of shear walls is complex and dependent on overturning moment vs. shear capacity ratios. If walls fail in shear, they are likely to degrade, and damage could be major and dangerous. • These buildings often have walls interrupted for large rooms or entrances. These discontinuous walls require special load-transfer details or can cause severe local damage. • Motels and hotels are often built with bearing walls and precast single-span concrete slabs. The bearing of these precast units on the walls and the ties to the walls can be a weakness, particularly if the floors do not have a cast-in-place fill over the precast slab units.	• Discontinuous walls can be mitigated by the introduction of new walls to create continuity, or by adding load-transfer elements at the point of discontinuity. • Wall can be changed from being "shear critical" (failing first in shear - usually bad) to "moment critical" (failing first in moment - usually acceptable) by adding layers of shotcrete or high-strength fiber and epoxy material. • All buildings with a relatively stiff and complete lateral force-resisting system, brittle or not, are candidates for seismic isolation. • In buildings with precast slabs, the bearing of the slabs can be improved by adding a steel or concrete bracket at each wall bearing. Diaphragms can be improved by adding a thin concrete fill or by "stitching" the joints with steel or fiber-epoxy material from beneath.

Table 8-3: Common deficiences by category of buildings 4

Category	Physical Characteristic	Performance Characteristics	Common Retrofit Techniques
EMA Model Building Types	C2 Concrete Shear Walls Type 2	• This building type could also have discontinuous walls similar to Shear Wall Type 1 buildings, with the same result. • These buildings generally have fewer walls than Type 1 and may be understrength, suffering damage to all the walls. • If the walls are not located symmetrically, torsion could result.	• Similar to Shear Wall Type 1, discontinuous shears can be locally retrofit. • Walls can be added to resist torsion and reduce the overall demand on all walls. • Individual walls can be changed from shear critical to moment critical. • All buildings with a relatively stiff and complete lateral force-resisting system, brittle or not, are candidates for seismic isolation.
	C3 Concrete w/ Infill Masonry	• These buildings are similar to Type S5, except that their concrete columns are more likely to fail the interaction with the masonry than the steel columns in building type S5	• See Building Type S5.
	PC1 Tilt-Up Concrete Walls	• The classic tilt-up failure consists of the exterior concrete walls pulling away from the roof structure, sometimes causing local collapse of the roof. • Less likely, but possible, is a shear failure within the panel piers due to large window openings.	• Retrofit measures for tilt-ups are well documented in mandatory retrofit ordinances passed in many jurisdictions in California. These requirements consist of creating an adequate connection between panels and roof structure, includings local connector, and transfers from the individual joists and purlins to the diaphragm. In some cases, diaphragms require strengthening or new braces are introduced in the center of the building to reduce diaphragm stress.
	PC2 Precast Concrete Walls	• The issue with most precast construction is the connections. Cast-in-place connections have generally performed well while welded ones have not. • Precast walls have seldom been used in regions of high seismicity until recently, when strict code requirements applied. Older precast wall systems may have problems with connections and may be hazardous, particularly if the walls are load bearing.	• Connections must be reinforced to ensure that the structure does not break apart. • It may be difficult to retrofit connections to provide an adequate seismic system, and new elements may also be required. • All buildings with a relatively stiff and complete lateral force-resisting system, brittle or not, are candidates for seismic isolation.
	PC2A Precast Concrete Frames	• Older precast frames only exist in regions of moderate or low seismicity. Adequate precast frame systems for high seismicity were only developed and built starting in about 2000. • Older frame structures may have inadequate connections and ductility, and if the structure starts to break apart, partial or complete collapse could follow.	• Connections must be reinforced to ensure that the structure does not break apart. • New elements such as concrete shear walls or steel braced frames will probably be required.
	RM1 Rein. Masonry Bearing Walls—Flexible Diaphragms	These buildings are much like tilt-up buildings, with the main weakness likely to be in the wall-to-floor or roof tie. Failure of these ties could lead to local collapse.	Similar to PC1 Tilt-up.
	RM2 Rein Masonry Bearing walls—Rigid diaphragm	This building is similar to C2 Concrete Shear Wall Type 2.	See C2, Type 2.

Table 8-3: Common deficiencies by category of buildings 5

Category	Physical Characteristic	Performance Characteristics	Common Retrofit Techniques
FEMA Model Building Types	URM Unreinforced Masonry Bearing Walls—Flexible Diaphragm	• The primary and most dangerous failure mode is the separation of the exterior wall from the floor/roof and falling outward. Local collapse of floors may follow. In smaller shaking, parapets often fail and fall outward onto the street or adjacent buildings. • The URM walls also often fail in shear with characteristic X cracking. In long-duration shaking, these walls may degrade and lose both lateral and vertical load-carrying ability.	• Several prescriptive ordinances are available to guide evaluation and retrofit. • Partial retrofit ordinances requiring parapet bracing, and/or new wall ties have also been used. • Typical retrofits are to add shear walls or steel braced frames to reduce demand on the URM walls, and to replace the URM wall-to-floor/roof ties and strength shear transfer at these locations. Sometimes, wood diaphragms also need strengthening that can be done by adding plywood.
	URMA URM with rigid diaphragms	• This building is similar to URM but will have a shallow arched masonry floor system with wood or concrete overlay, or a concrete slab floor. Flat arched floors have no proven ability to act as a diaphragm and could lead to failures of exterior walls similar to URM, although the ensuing failure of the floors could be much more dangerous. • Concrete slabs, if tied well to the URM walls, may force high in-plane load into the URM walls and cause shear failures. If URM walls are relatively solid, this building type may not form a dangerous collapse hazard.	• Similar to URM, a complete lateral load system must be created to resist out-of-plane URM wall loads and distribute them, through the floor/roof diaphragm to perpendicular walls. Flat arched floors must, at a minimum, be tied together, and cross-building ties must be installed. • URM walls must be adequate for in-plane forces or new elements added. • All buildings with a relatively stiff and complete lateral force-resisting system, brittle or not, are candidates for seismic isolation.

8.8.2 To Learn More

Applied Technology Council, *Seismic Evaluation and Retrofit of Concrete Buildings*, Report No. SSC 96-01, California Seismic Safety Commission, 1996.

A contemporary of FEMA 356 that features many of the same methods and performance terminology. This document contains a good description of retrofit strategies.

Earthquake Engineering Research Institute *Ad Hoc Committee on Seismic Performance, Expected Seismic Performance of Buildings, SP 10*, Earthquake Engineering Research Institute, 1994.

This document was published slightly before FEMA 356 and thus contains slightly different performance terminology. However, photo examples and extensive description of damage states are contained. In addition, estimates are given for the approximate number of various buildings that would be expected to be in various damage states for different ground motion intensities.

Federal Emergency Management Agency, Region 1, *Safeguarding Your Historic Site*, Boston, MA.

This document contains an extensive bibliography covering renovation and repair of existing buildings.

Freeman, John R. *Earthquake Damage and Earthquake Insurance.* McGraw-Hill Book Company, Inc., 1932, New York.

Extremely interesting from a history standpoint, this book Includes discussion of seismology, geotechnical engineering, structural engineering, codes, and loss estimation, and excellent history and available data on earthquakes up to 1932.

Holmes, William T., *Risk Assessment and Retrofit of Existing Buildings*, Proceedings Twelve World Conference on Earthquake Engineering, Auckland, New Zealand, 2000.

This paper contains a more technically oriented description of the methods of FEMA 356 and strategies for design of retrofit systems.