NIST GCR 17-917-44

Seismic Analysis, Design, and Installation of Nonstructural Components and Systems – Background and Recommendations for Future Work

Applied Technology Council

This publication is available free of charge from:
https://doi.org/10.6028/NIST.GCR.17-917-44

National Institute of Standards and Technology
U.S. Department of Commerce

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Cover image – Top: Attachment failure of hung architectural, mechanical, electrical and plumbing systems within the Santiago airport terminal in the 2010 Chile earthquake (Photo Credit: Eduardo Miranda, Stanford University). Middle: Failure of compressor mounted on vibration isolators in the 1994 Northridge earthquake (Photo Credit: Wiss, Janney, Elstner Associates). Bottom: Severe damage to exterior curtain wall in the 2014 South Napa earthquake (Photo Credit: Maryann Phipps, Estructure).

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Prepared for
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https://doi.org/10.6028/NIST.GCR.17-917-44

March 2017

U.S. Department of Commerce
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NIST GCR 17-917-44

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Preface

In 2013, the National Institute of Standards and Technology (NIST) awarded the Applied Technology Council (ATC), a National Earthquake Hazards Reduction Program (NEHRP) "Earthquake Structural and Engineering Research" task order contract (SB1341-13-CQ-0009) to conduct a variety of tacks. In 2014, NIST initiated Task Order 14-491, entitled "Seismic Analysis and Design of Nonstructural Components and Systems." The objective of this task order was to improve the seismic design of nonstructural systems and components in the areas that will have the largest impact to public safety and economic welfare, with an emphasis on construction regulated by building codes.

This project was conceived in direct response to recommendations provided in the NIST GCR 13-917-23, *Development of NIST Measurement Science R&D Roadmap: Earthquake Risk Reduction in Buildings* (NIST, 2013). In particular, the GCR 13-917-23 report identified nonstructural issues as a top priority, calling for problem-focused studies in critical areas related to nonstructural design criteria. This report provides a summary of the first phase of work, which included background knowledge investigations, a workshop to identify current challenges faced with nonstructural code provisions, and development of recommendations for future studies and research.

The Applied Technology Council is indebted to the leadership of Maryann Phipps, Project Director, and to members of the Project Technical Committee consisting of Saeed Fathali, John Gillengerten, Tara Hutchinson, and Ricardo Medina, for their contributions in developing this report and the resulting recommendations. Background information and technical assistance was provided to the project by Bill Holmes, Bob Pekelnicky, Roy Lobo, John Silva, and Xiang Wang. The Project Review Panel, consisting of Bob Bachman (chair), Doug Honegger, Mike Mahoney (ex officio member), Eduardo Miranda, Keri Ryan and Derrick Watkins, provided technical review and comment at key developmental stages of the project. An invited workshop consisting of industry experts involved in the design, manufacture, installation, and code enforcement of nonstructural components and systems was convened to obtain input on the application of nonstructural seismic design provisions in current design and construction practice. The names and affiliations of those who participated in the workshop and all who contributed to this report are provided in the list of Project Participants at the end of this report.

The Applied Technology Council also gratefully acknowledges Steven L. McCabe (NIST Program Manager and Contracting Officer's Representative) and Matthew Hoehler (NIST Project Manager and Technical Point of Contact) for their input and guidance in the preparation of this report, John Kunz for workshop facilitation services, and Carrie Perna for ATC report production services.

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Chapter 1

Introduction

Nonstructural components and systems account for the majority of direct property losses due to earthquake damage. Although significant structural damage to modern buildings has generally been rare in moderately strong earthquakes, costly and disruptive nonstructural damage is much more widespread, and can result in additional economic losses from functionality and business interruptions. The National Institute of Standards and Technology (NIST) GCR 13-917-23 report, *Development of NIST Measurement Science R&D Roadmap: Earthquake Risk Reduction in Buildings* (NIST, 2013) identified nonstructural issues as a top priority, calling for problem-focused studies on four critical areas related to nonstructural design criteria: (1) the vertical distribution of nonstructural design forces over the height of a building, F_p; (2) the response modification coefficients for nonstructural components, R_p; (3) the overstrength factors used in the design of nonstructural anchorage; and (4) nonstructural component and system performance metrics.

1.1 Project Objectives and Scope

The NIST-sponsored ATC-120 Project, *Seismic Analysis and Design of Nonstructural Components and Systems*, was a direct result of the recommendations contained in the NIST GCR 13-917-23 report. The purpose of this project was to improve technical aspects of nonstructural system design in the areas that will have the largest impact to public safety and economic welfare, with an emphasis on construction regulated by building codes. The resulting recommendations are intended to have practical application to the most common types of structures, and be conceived in a manner that facilitates ease of implementation throughout areas of the country with significant seismic hazard.

With recent advancements in performance-based design methodologies, and the development of concepts for community resilience, there has been an increased emphasis on economic losses resulting from downtime and loss of building function due to poor seismic performance of nonstructural components and systems, and an increase in research and testing to improve the seismic performance of nonstructural components and systems. Development of new research results, and the availability of new international standards, makes detailed examination of U.S. nonstructural seismic design and construction practices possible at this time.

Work was split into phases. The first phase of work included a detailed investigation of the state of knowledge related to the performance of nonstructural components and systems in past earthquakes, the history and evolution of nonstructural seismic design provisions and criteria, and currently available information on research and testing. A workshop consisting of invited industry experts involved in the design, manufacture, installation, and code enforcement of nonstructural components and systems was convened to obtain broad-based industry input on the application of nonstructural seismic design provisions in current design and construction practice.

An emphasis of the background knowledge investigation and workshop activities was to determine if a disconnect exists between current design requirements and observed (or expected) performance in earthquakes. Where significant gaps or opportunities for improvement were identified, problem-focused studies and additional research have been recommended for implementation in future phases of work.

1.2 Report Organization and Content

This report provides a summary of findings from the first phase of work, including background knowledge investigations, conclusions from workshop discussions, and recommendations for future studies and research. It collects and summarizes the body of available knowledge on seismic performance observations, analytical studies, testing programs, and practice issues related to nonstructural components. Intended to be used as a foundation for future advancement, this report does not explicitly include all past research and development, but it describes the context for current code requirements and selectively highlights areas where additional technical information is needed for improvement of seismic code provisions.

Chapter 2 provides an overview of the performance of nonstructural components in selected earthquakes over the last fifty years, including general observations on the usefulness of the available quantitative and qualitative performance data in post-earthquake reconnaissance efforts.

Chapter 3 discusses the history and development of design force requirements for nonstructural components and anchorage in the United States and other countries.

Chapter 4 summarizes industry standards applicable to nonstructural components, with a discussion of how they are implemented within nonstructural design requirements.

Chapter 5 presents available guidelines for the design and installation of nonstructural components and systems, as well as criteria for performance-based design tools that have been developed.

Chapter 6 summarizes methods that have been developed to measure response and validate the performance of nonstructural components and systems within larger controlled experiments, such as shake table testing of full-scale structures, and component testing for seismic qualification.

Chapter 7 presents active areas of analytical research related to the development of floor response spectra, their relationship to existing equations for static design forces, and possible use in the design of nonstructural components and systems.

Chapter 8 summarizes findings and conclusions related to current issues in nonstructural component and system design and construction practice based on workshop discussions and studies available in the literature.

Chapter 9 provides a summary of overall findings from the first phase of work and outlines a program of recommended problem-focused studies intended to fill gaps in available knowledge, improve nonstructural design and construction practice, and improve the performance of nonstructural components and systems in earthquakes.

Appendix A provides materials that were used in workshop discussions to discuss acceptable performance of nonstructural components and systems, given different scenarios related to component or system type, earthquake shaking intensity, and building design level.

Chapter 2

Observed Performance of Nonstructural Components

Many of the advancements in seismic-resistant design have stemmed from post-earthquake observations of damage, and analysis of the root causes of that damage. In general, the emphasis has been on describing and analyzing earthquake damage to the primary seismic force-resisting system of the structure. In-depth studies of the performance of seismic force-resisting systems, the contribution of gravity load-carrying systems, and the influence of the design and detailing of elements of the structural system are numerous. In contrast, performance of nonstructural components and systems, which is often the primary source of losses in seismic events, is usually discussed only in general terms, if at all.

There are hundreds of published articles and reports that discuss seismic performance of nonstructural components in some level of detail. This chapter summarizes the findings of a sampling of post-earthquake reports that provide more in-depth treatment of nonstructural issues. These examples are only representative, as the body of work on post-earthquake investigations is large. The selected examples do, however, provide insight into how nonstructural performance is presented in.

There is a tendency to minimize the potential safety risks posed by nonstructural damage, based on the assumption that the vast majority of deaths and serious injuries in earthquakes will be the result of total or partial building collapses. This belief is supported by low casualty figures in earthquakes that have occurred in United States. These low casualty figures may be due, in part, to chance. The 1964 Great Alaska earthquake occurred at 5:36 PM on Good Friday. Most people where either home or on their way home and few were in schools and businesses. The 1971 San Fernando and 1994 Northridge Earthquakes both occurred in the early morning hours, when schools and businesses were empty. Had either of these earthquakes occurred at mid-day during the work week when schools and businesses were crowded, casualties due to nonstructural damage would have been much higher.

With the exception of the 1964 Great Alaska and 1971 San Fernando earthquakes, the focus of this chapter is on seismic events that impacted regions where nonstructural component anchorage and bracing requirements have been adopted for new construction, and some level of enforcement of the requirements has occurred. Earthquakes within California (and the United States) have been emphasized because nonstructural anchorage and bracing requirements were not enforced in most other parts of the country (or the world). For example, the 2001 Nisqually earthquake (magnitude 6.8) resulted in widespread damage in the Puget Sound area, but nonstructural components in many newer buildings were not designed for seismic forces. The 2011 Virginia earthquake (magnitude 5.8) struck a region with little historical seismic activity and where design for seismic forces for many types of nonstructural components is generally not required. Internationally, the strong earthquakes that struck the Christchurch, New Zealand area in 2010 and 2011 caused extensive damage, but few of the buildings had braced nonstructural components and systems. The 2010 Chile earthquake, however, provides an international opportunity to study the performance of braced nonstructural components because Chile has nonstructural design provisions that meet or exceed those in the United States, although enforcement is somewhat inconsistent.

An increasing number of new and existing buildings have been fitted with strong motion recording instruments. Hundreds of these buildings have experienced one or more earthquakes, presenting the opportunity to correlate nonstructural performance to known shaking intensity. Two of the reports included in this chapter are focused on the response of instrumented buildings: the Santa Clara County Administration Building in the 1989 Loma Prieta earthquake, and the Olive View Medical Center in the 1994 Northridge earthquake. These reports illustrate the types of nonstructural performance information that can be generated when actual earthquake demands, component performance, and design information are known. Unfortunately, studies that focus on nonstructural performance of instrumented buildings are rare. Even so, a great deal has been learned after studying the floor acceleration and story drift behavior of instrumented buildings in earthquakes, and data from these buildings form the basis of the standard component force equation in current seismic standards.

2.1 March 27, 1964 Great Alaska Earthquake

Although the seismic performance of structures in the magnitude-9.2 Great Alaska earthquake on March 27, 1964 was the subject of many studies, one report in particular provided a comprehensive examination on the performance of nonstructural components. In the aftermath of the earthquake, a comprehensive study produced *A Report on Non-Structural Damage to Buildings - Alaska Earthquake, March 27, 1964* (Ayres et al., 1967). This report, completed in 1967, was prepared for the U.S. Army Engineers District in Anchorage, Alaska at the request of the Engineering Panel of the National Academy of Sciences Committee on the Alaska Earthquake. It was subsequently made broadly available by the Consulting Engineers Association of California in 1971. This 460-page report includes extensive descriptions of the performance of a wide range of nonstructural components, including elevators, mechanical systems, lighting fixtures, electrical systems, emergency power and lighting systems, exits, façades and glazing, ceilings, partitions, storage racks, and furniture. With the exception of fire protection sprinkler systems, none of the nonstructural components were designed considering earthquakes.

This report was prepared by a multi-disciplinary team that consisted of an electrical engineer and two mechanical engineers, one of whom had additional specialized training in structural engineering. The team was assisted by consulting architects and an elevator consultant. The scope of this report was limited to damaged nonstructural components in buildings in the Anchorage area, along with a few structures in Whittier, Alaska. This report was started almost two years after the earthquake, and relied heavily on photographs and studies by others, as well as interviews, surveys, and first-hand information from individuals involved in the repair efforts.

This report identifies patterns of damage to nonstructural components that continue to be observed today. Recommendations for changes to design and installation practice to improve performance were made by the authors. Damage observations found in this report are summarized below.

2.1.1 Nonstructural Component Design Process

A summary of the nonstructural design process in the mid-1960s indicates, as is still common today, the design of nonstructural components was performed by subcontractors with limited review by structural engineers or architects. The building code requirements at the time focused on minimum standards to safeguard life. Seismic performance for most nonstructural components was considered a property damage issue rather than a life safety issue, and not covered in the 1961 *Uniform Building Code* (UBC) (ICBO, 1961), which was the building code in force in Anchorage at the time of the earthquake. Details of many nonstructural components, including equipment, distribution systems, ceilings, light fixtures, and façades, were omitted from the design drawings and were installed in accordance with standard trade practice.

2.1.2 Elevators

At the time of the earthquake, there were approximately 100 elevators in the Anchorage area, about evenly split between traction and hydraulic types. Hydraulic elevators sustained minor damage, while damage to traction elevators was extensive. The authors reported that according to one major manufacturer, "earthquake survival, as such, is not a consideration in the design of our equipment or system." They also reported that among elevator repairmen working in Alaska, it was believed that earthquakes would always "knock out" elevator systems. Damage to traction elevators included anchorage failures of motor-generator sets and control panels, and guide rail failures that allowed counterweights to obstruct the shafts. Fortunately, power failures prevented the counterweights from colliding with the elevator cabs (cars).

It was estimated that 80% of the earthquake damage to elevator machinery was due to anchorage failures of the motor-generator sets, which were typically vibration isolated. Some units displaced up to eight feet. Elevator control panels suffered little damage when anchored to the floor, but unanchored units toppled. Control relay panels were sometimes damaged when unlatched hinged panels were thrown open during the earthquake. The authors reported that elevator machinery and control panels in the penthouses were subject to higher lateral forces than similar equipment in basements, citing undamaged traction elevator equipment that was placed in the basement of a three-story structure.

Counterweight guiderail failures occurred in almost all traction elevators, with bent guiderails and broken roller guide assemblies observed. In some cases, the guiderails tore free from the structure, and swinging counterweights damaged spreader beams and conduit in the shafts. The guide rails for the cars themselves sustained little damage. Several of the buildings had unreinforced concrete masonry unit (CMU) shaft walls, and falling debris from the shaft walls was reported.

Damage to the approximately 50 hydraulic elevators in the Anchorage area was very minor, with some units placed back into service once electrical power was restored. Only one case of damage to hydraulic elevator equipment was reported, and it was attributed to building settlement. Damage to shaft walls prevented the use of another hydraulic elevator.

2.1.3 Mechanical Systems

The 1967 Ayres report classified a wide range of nonstructural components as mechanical systems, including boilers, furnaces, flues, chimneys, plumbing, piping, fans, ducts, refrigeration compressors, heating, ventilating and air conditioning systems, tanks, fire sprinklers, and gas systems. Few buildings in the Anchorage area had air-conditioning systems. The only piping system with lateral bracing was fire sprinklers, which were governed by the installation requirements of the National Board of Fire Underwriters.

Piping Systems

The report described the performance of piping systems by function, pipe materials, and fitting type. Many piping failures occurred at fittings and threaded fittings seem to be more vulnerable to damage than brazed or welded connections. Most pipe failures were attributed to large displacements of unbraced pipe. Failures in screwed joints often occurred where long unbraced horizontal runs of pipe joined short vertical risers, or at horizontal changes of direction, or where connected to equipment.

Piping damage was often reported in large, low-rise structures such as warehouses, which had long horizontal runs of unbraced piping. In taller buildings, less piping damage was observed in the lower levels. Small branch lines that were closely attached to the building structure failed at connections to unbraced horizontal piping mains. Failures of hangers were observed and sections of piping fell. Some damage to utility connections was observed due to differential settlement, and broken underground fuel lines to emergency generators were reported. Unbraced small diameter piping such as vacuum and compressed air lines generally performed well, although damage was observed where the piping connected to equipment. Little damage to vertical risers was observed. Although flexible connections were provided for piping running across seismic joints, they had not been designed for relative seismic motions, and damage was observed.

Fire sprinkler piping suffered little damage with good performance being attributed to lateral bracing. In a school gymnasium, a sprinkler head was activated when it struck a roof member. Sprinkler heads in unbraced ceilings cut the acoustical tiles due to differential movement with some cuts reported to exceed one foot in length.

Tanks

Most tanks were unanchored, and widespread damage to them was reported. Sand filter, water softener, domestic hot water, heating hot water expansion, and cold water storage tanks shifted, toppled, or rolled over when they were inadequately anchored or when tank legs failed. Shifting or toppling of tanks ruptured the connected piping. Failure of some large tanks resulted in substantial consequential damage to adjacent components. Some tanks failed due to an incomplete load path. A saddle mounted tank on the roof of the fourteen-story Hodge Building in Whittier, Alaska shifted five inches, shearing the water lines because the steel saddles were anchored to concrete piers, but the tank was not connected to the saddles. Similar damage was reported at other buildings. In the penthouse of the fourteen-story Mt. McKinley Building in Anchorage, Alaska, the basement plumbing and heating equipment was undamaged, but an unanchored hot water expansion tank in the penthouse slid approximately ten feet, causing water damage in the lower floors.

Mechanical Equipment and Ductwork

Damage to air handling equipment was limited and most flexible duct connectors at fans were torn, and supports and hangers damaged. Fans and their motors and drives were undamaged, although unanchored fans often displaced and rotated. Unbraced suspended mechanical units swung during the earthquake, fracturing piping connections and crushing the connecting ductwork. Many unbraced suspended gas unit heaters fell. An anchored air-conditioning unit at a medical building southeast of downtown Anchorage, slid two feet when the anchor bolts sheared off, damaging the connecting pipes and ducts. In the same facility, a suspended exhaust fan fell when the hanger rods failed. Spring vibration isolators were identified as the weak link in the load path between mechanical components and the structure, and dynamic amplification of lateral loads for vibration isolated components was observed.

Most sheet metal ductwork suffered little damage, although some horizontal runs of ductworks at a medical facility were found resting on the corridor ceilings after the earthquake, when the hanger attachments pulled out of the slab above.

Boilers, Plumbing, Vents, Stacks, and Chimneys

Most heating boilers were located in the basements of buildings. Although some packaged boilers shifted a few inches and some vent connectors were damaged, the equipment was usable once gas service was restored. Hundreds of small unbraced gas-fired and electric domestic water heaters toppled, and vent connectors were damaged. The legs buckled under the some of the water heaters. Damage to plumbing fixtures was limited to those struck by falling debris, and vertical plumbing stacks in tall buildings were practically undamaged.

Some unreinforced masonry stacks were cracked, and in some cases the top part of the stacks collapsed and fell onto the building. Many prefabricated metal stacks with heavy refractory liners collapsed, while lightweight double wall sheet metal vents and stacks suffered little damaged. Many masonry and stone chimneys in residences and low-rise commercial buildings were damaged.

2.1.4 Architectural Components

The 1967 Ayres report summarized the performance of a variety of architectural components, including means of egress (exits), façades and glazing, ceilings, partitions, storage racks, and contents.

Many exit corridors and stairwells were blocked by falling debris from damaged unreinforced CMU walls. In one case, the debris covered an entire exit stairway, making escape impossible. Stairwells with stud walls covered with lath and plaster cracked extensively, but exits were still useable. Egress paths were blocked by fallen light fixtures and ceiling tiles. Most seismic joints between adjacent structures were generally detailed for longitudinal (perpendicular to joint) movements only, and relative lateral (parallel to joint) movements resulted in extensive damage. Exit doors were jammed due to structural damage around the door frames. Many doors were blocked by displaced furniture and building contents.

Lightweight metal and glass curtain walls performed well, compared to buildings with heavy masonry façades or with walls faced with brick. This report observed the importance of considering displacement compatibility between the building seismic force-resisting system and the exterior façade in design. The five-story reinforced concrete J.C. Penney Building in Anchorage suffered heavy damage caused, in part, by torsional response induced by an irregularity due to heavy precast concrete cladding panels installed on the north and east façades of the building. During the earthquake, as shown in Figure 2-1, some of the precast panels fell. Two people were reported to have been killed by the falling panels.

A number of concrete frame buildings with masonry infill walls suffered damage to both the walls and frame elements. No provisions had been made to isolate the walls from the lateral displacements of the frame.

Figure 2-1 People escaping the J.C. Penney store during the 1964 Great Alaska earthquake (NISEE, 2016a).

Most ceilings in Anchorage were suspended lath and plaster or gypsum board, but suspended acoustical tile systems were used in newer buildings. The plaster and gypsum board ceilings suffered little damage, while many suspended acoustical tile ceilings were heavily damaged, especially at the perimeters of the ceiling. Light fixtures fell from damaged acoustical tile ceilings. Unreinforced CMU partitions were widely used and were badly damaged in the earthquake. Lath and plaster partitions were badly cracked, while partitions of stud and drywall construction sustained only minor damage.

The authors of the 1967 Ayres report expressed surprise that more people were not injured by falling objects. Heavy furniture toppled and cabinets lost their contents. Book shelves lost their contents early in the shaking and as a result many suffered little damage. Industrial storage racks shifted and toppled and many lost their contents.

2.2 February 9, 1971 San Fernando Earthquake

The 1971 San Fernando earthquake caused significant damage to a large number of modern structures in the greater Los Angeles area, especially in the San Fernando Valley of Southern California. Reports published following the magnitude-6.6 earthquake tended to focus on structural performance and damage. Two studies are examined: the first is a general report that covers engineering aspects of the earthquake, and the second is a study similar to the nonstructural report for the 1964 Great Alaska earthquake, but of reduced scope.

2.2.1 Engineering Aspects

Representative of reports published immediately after an earthquake is *Engineering Aspects of the 1971 San Fernando Earthquake* (Lew et al., 1971). This 419-page report covers a range of topics and includes case histories of damage to a variety of structures. There were no strong motion instruments in the vicinity of some of the hardest hit buildings included in the report, but estimates of the ground shaking intensity are included.

Medical facilities in the San Fernando Valley were hit particularly hard in the earthquake. The new 850-bed Los Angeles Olive View Medical Center had been dedicated in November 1970, just months prior to the earthquake. The reinforced concrete building was constructed under the 1965 County of Los Angeles Building Code (1965). Most of the buildings on the site suffered severe structural damage, which dominated the attention of the engineering community. Horizontal ground accelerations might have exceeded 0.5g. The report contains little information on the performance of nonstructural components at the site, although it is implied that damage was substantial. Severe story drift in the first story of the main building destroyed the curtain walls, partitions, and elevator doors at that level. Photographs of other areas on the ground floor show extensive damage to food service equipment. Severe damage to the equipment in the power plant building was reported with boilers shifting up to four feet. An example of this damage is shown in Figure 2-2.

Figure 2-2 Broken piping in the new power house for the Olive View Hospital, 1971 San Fernando earthquake (NISEE, 2016b).

Holy Cross Hospital was a ten-year old facility, which included a seven-story reinforced concrete main building, a service building (mechanical plant), and a single-story continuing care facility. The main building suffered significant damage. The elevators were inoperable due to bent rails and displaced counterweights, so the 170 patients were evacuated via an exterior stairway. The report suggests the emergency power system functioned during the evacuation. The report references a photograph of overturned shelves on the seventh floor of the main building, suggesting that shaking at the upper levels of the building was severe. Some equipment in the service building displaced. The six-story Indian Hills Medical Center building adjacent to the hospital suffered some structural damage to concrete shear walls. The steel stairs in this building were damaged when the connections to the concrete walls at the landings failed. The stairs were “shaky” but usable. The elevators were inoperable following the earthquake.

Pacoima Memorial Lutheran Hospital was built in 1959, consisting of a reinforced concrete structure with one-, two-, and three-story sections. Initially, 100 patients were evacuated from the damaged facility, but the emergency facilities were reopened out of necessity. Initially, the two elevators were inoperative, but one elevator was later returned to service. The facility operated on emergency power. A stairwell in the center of the hospital was damaged extensively. The report refers to photographs of the building exterior that indicate some loss of glazing.

The Sepulveda Veterans Administrative Hospital was opened in 1955. The buildings on the site ranged from one to six stories, with reinforced concrete frames and reinforced masonry walls. Overall, there was minimal structural damage to the facility. Damage to the elevator guide rails was reported, and some damage was reported to the hoisting machinery. Extensive repairs to the partitions and seismic joints were necessary.

Northridge Hospital, located about fifteen miles from the epicenter, was a five-story steel structure with masonry shear walls and clad with brick veneer. Although damage to the brick veneer was extensive, all mechanical and electrical equipment was reported as functional, including the elevators.

Lightweight concrete shear walls within the ten-story Kaiser Foundation Hospital suffered significant damage. Doors to the stairwells at the second and third floors were inoperable due to damage to concrete spandrel beams. The emergency power system functioned and the hospital remained in operation. All the elevators were inoperable following the earthquake. About fifty glass panels fell from the building in the earthquake, and many additional windows subsequently were blown out by wind. The wind damage was attributed to weakening of the glazing units in the earthquake.

The seven-story Holiday Inn in Panorama City was fitted with strong motion instruments at the first floor, fourth floor, and roof. A maximum horizontal acceleration of 0.28g was recorded at the first floor and 0.39g was recorded at the roof. Nonstructural damage was most pronounced at the second, third, and fourth floors, with partition damage being noted.

The thirteen-story Union Bank Building was located across the street from an instrumented structure, where a peak horizontal ground acceleration of 0.23g was recorded. The concrete moment frame building suffered jammed doors on the second, third, fourth, and fifth floors, connection failures at the landings of the steel stairs, some partition damage, and cracked glass and veneer at the first floor. The elevators were inoperable following the earthquake.

Postal facilities in the San Fernando Valley reported nonstructural damage, with suspended light fixtures falling due to failures at connections and acoustical tiles falling from suspended ceilings. Unrestrained battery-operated emergency lighting fell to the floor, and fire extinguishers were thrown from their brackets.

2.2.2 *Nonstructural Damage*

Following the 1971 San Fernando earthquake, the National Oceanic and Atmospheric Administration hired J.M. Ayres, who also spearheaded the effort to collect data on nonstructural component performance following the 1964 Great Alaska earthquake, to lead another study to evaluate damage to nonstructural components and systems. The report, *Nonstructural Damage, The San Fernando, California Earthquake of February 9, 1971*, written by Ayres and Sun (1973) summarized damage to equipment, but did not consider other nonstructural components and systems.

The report includes a breakdown of substantial damage to elevators. State and local government agencies regulating elevators requested earthquake damage reports from twelve elevator companies, Los Angeles County, and the University of California, Los Angeles. The information shows widespread damage to traction elevators, including damage in areas that experienced low ground shaking intensities. The most significant damage was due to displaced counterweights, and their impact on cars and other equipment. Counterweights where thrown out of their guide rails in 674 elevators, and the counterweights struck the cars in 109 instances. In many cases, damage to the cars occurred after the earthquake when the elevators were restarted, and maintenance personnel were unaware that the counterweights were loose in the hoistway. Guide rail brackets were broken or damaged in 174 cases, and guiderails were out of alignment, bent, or broken in 49 cases. Motor generators that were vibration isolated and not anchored to the floor were thrown from their mounts and damaged. The drive motors and traction machines were generally anchored to the floor and sustained little damage. Selector and controller panels suffered little damage when they were anchored to the floor or braced to the machine room wall. The authors repeatedly expressed concern regarding the increased lateral flexibility of modern buildings. Building story drift damaged hoistway walls, dropping debris in the shafts and in two cases the walls bowed far enough into the shafts to strike the cars. Building story drift also resulted in jammed and misaligned elevator doors. Hydraulic elevators performed well. Damage was limited to unanchored pumps and oil tanks that shifted during the earthquake, and hydraulic cylinder casings that leaked or remained out of plumb.

With the exception of fire sprinkler piping, little consideration was given to seismic resistance of nonstructural components and systems. Many mechanical components survived the earthquake with only minor damage, although unanchored components shifted and often rupturing pipe and duct connections. Non-isolated components that were well-anchored to the floor sustained little or no damage. Most of the damage to mechanical components was attributed to failures of vibration isolators. Heavy components on vibration isolators performed poorly throughout the Los Angeles area, especially in high-rise buildings. Damage was not limited to areas of high shaking intensity. Amplification of earthquake motions in the upper levels of structures was observed.

The equipment and systems in the central plant at Olive View Hospital were damaged, although the building itself sustained only minor structural damage. Vibration isolated components were damaged and unanchored components displaced about three feet. The roof-mounted cooling towers were racked, and the boiler stacks were tilted when the large, unanchored packaged boilers displaced up to 3½ feet. Heavy tanks containing liquids were damaged. Unanchored tanks shifted or toppled, and legs and supports failed. Tanks legs made up of pipe with screwed fittings failed at the threaded joints.

The Ayres and Sun report indicates that piping systems sustained minor damage. Where damage occurred, it was attributed to excessive pipe displacement and differential movement at connections to equipment. Piping with welded, brazed or soldered joints was generally undamaged, while failures were observed at screwed fittings. Damage was observed at seismic separations between buildings, since the pipe was not designed for the differential displacements.

Fire sprinkler systems were reported to have performed well. References to data collected by the Pacific Fire Rating Bureau indicated that, of the 973 buildings with fire sprinkler systems located in the area of strongest shaking, 62 buildings were damaged and only 38 of these suffered "slight to severe" damage to their sprinkler systems. Failures of C-clamps, lag bolts, and cast iron fittings were observed, as well as a few pipe failures. Sprinkler drops and heads damaged suspended ceilings, and some water damage occurred when sprinkler heads activated due to interaction with the ceiling or other elements.

Damage to air handling systems was considered minor. Damage to most ducts was limited to torn or loosened joints. However, when ducts and diffusers were not supported independent of the ceiling, they fell when the ceiling was seriously damaged. Suspended light fixtures were heavily damaged, and many fixtures weighing 40 to 80 pounds fell. It was suggested that serious injuries and fatalities were avoided only because most buildings were unoccupied at the time of the earthquake. Surface mounted recessed light fixtures were practically undamaged, and pendant light fixtures suffered extensive failures, especially in schools.

2.3 October 1, 1987 Whittier Narrows Earthquake

The 1987 Whittier Narrows earthquake, a moderate magnitude-5.9 event, was centered in a highly urbanized region, ten miles east of downtown Los Angeles. The California State University Los Angeles (CSULA) campus, located six miles from the epicenter, sustained over $20 million in damage, and one student was killed by a falling architectural precast concrete panel. An article detailing the damage sustained at the campus by Taly (1988) appeared in the May 1988 issue of *Earthquake Spectra*. The buildings on the campus were constructed from late 1950s through the 1970s, and sustained some structural damage and significant nonstructural damage. Some of the more significant damage described in the article is summarized below.

The eight-story Administration Building was outfitted with strong motion instruments that recorded horizontal ground shaking of 0.3g at the ground level and 0.48g at the roof. The building, a composite steel and reinforced concrete structure with a soft first-story, suffered cracking in partition walls, and damage to acoustical tile ceilings and suspended light fixtures. Windows in the glazed curtain wall cracked, and two precast exterior sunshades were dislodged.

In the eight-story Physical Sciences Building, a reinforced concrete shear wall structure, all five passenger elevators were out of service, and the freight elevator was damaged when the counterweights crashed into the elevator cab. Chemical spills in the labs on the top three floors of the building required response from hazardous materials (HazMat) teams. Countertop lab equipment fell to the floor and was damaged, and supplies were dumped from shelving. A flash fire occurred in a lab on the eighth floor, destroying the lab and causing smoke damage over about a quarter of the eighth floor along with water damage over portions of the sixth, seventh, and eighth floors. In the penthouse, the supports of a chiller failed and the unit displaced, breaking pipes and causing water damage. Prior to the earthquake, the building was known to contain friable asbestos, and tests following the earthquake showed unacceptable levels of asbestos on the top floor and penthouse, forcing their closure.

The three-story Salazar Hall classroom building, a steel-frame structure with reinforced concrete shear walls, was the most severely damaged building on the campus, suffering serious structural and nonstructural damage. Approximately 75% of the suspended ceiling grid failed, with damage to fluorescent lights and some conduit. Some partition walls were heavily damaged, in one case a wall displaced four inches horizontally at the ceiling, perpendicular to the wall. Some desktop computers were damaged by falling ceiling components. Wall mounted televisions fell and some water leaks were noted. As with other buildings at CSULA, the building was known to contain friable asbestos, and tests following the earthquake showed unacceptable levels of the substance.

The John F. Kennedy Memorial Library, two four-story reinforced concrete buildings connected by a four-level reinforced concrete link bridge, experienced little structural damage but severe nonstructural damage. The link bridge suffered severe structural damage. In the penthouse of one building, the vibration isolation mounts of two chillers failed, breaking the water pipes, which resulted in flooding. Although they were braced, most of the metal bookshelves in the library failed due to buckling in light-gauge steel bracing at the top of the shelving units and failures in the rod bracing. About 40% of the book collection fell to floor. In the link bridge, forty 2-foot by 13-foot windows broke, scattering glass in the immediate vicinity. The seismic separation joints between the bridge and the library buildings suffered severe pounding damage.

The parking structure was a two-level reinforced concrete and prestressed concrete structure. Precast spandrel panels of lightweight concrete, six inches thick, measuring seven to 20 feet in length, and over seven feet in height, were installed along the perimeter of the parking deck. The units were attached to prestressed concrete L-beams with four to eight connections depending on the length of the panel. The connections on a twelve-foot long panel failed, causing the panel to fall to the ground (Figure 2-3). This failure resulted in the death of one student. In an aftershock, concrete inserts supporting a twelve-inch diameter chilled water line in the parking structure failed, and a 125-foot long section of the pipe fell.

Figure 2-3 Fallen precast concrete panel at the California State University Los Angeles parking structure, 1987 Whittier Narrows earthquake (Çelebi et al., 1987).

2.4 October 17, 1989 Loma Prieta Earthquake

The 1989 Loma Prieta earthquake was centered south of the San Francisco Bay Area in the Santa Cruz Mountains of Northern California. Lasting less than fifteen seconds, the magnitude-6.9 earthquake caused more than $7 billion in damage and killed 62 people. Many technical reports were published in the aftermath of the earthquake. Two studies are discussed: (1) an article covering nonstructural performance in a special issue of *Earthquake Spectra* covering the Loma Prieta earthquake; and (2) a report on the correlation between nonstructural performance and recorded building response in the Loma Prieta earthquake.

2.4.1 Nonstructural Damage

In the aftermath of the Loma Prieta earthquake, the Earthquake Engineering Research Institute (EERI) published a special edition of *Earthquake Spectra* focusing on different aspects of the earthquake. One of the articles, by Ding et al. (1990) focused on the impact of the earthquake on nonstructural components and systems. Reports of nonstructural damage were widespread and the severity of damage was influenced by proximity to the epicenter and local ground conditions. Some buildings constructed in accordance with seismic codes suffered nonstructural damage sufficient to limit access. The actual level of damage to individual structures was difficult to assess because much of the damage was not visible from the building exterior. Data in the article are general in nature, with limited specific descriptions of damage to individual structures.

No damage was noted to modern architectural precast panels, although it is noted elsewhere in the article that many buildings suffered substantial damage to "exterior panels of all types" that were, in some cases, on the verge of failure. The article states that some architectural spandrel panels acted as structural members, and that failures, in general, could be attributed to improper detailing for story drift. Damage to exterior glazing was observed throughout the region, and glazed storefronts in older buildings were particularly vulnerable. Although windows and glazed storefronts broke, the much-feared widespread lethal failures of cladding and glazing did not occur.

Damage to suspended acoustical ceiling systems was often observed at ceiling perimeters, and was attributed to the lack of splay-wire bracing. At the Geary Theater in San Francisco, as shown in Figure 2-4, a portion of the ceiling and stage lighting grid collapsed on the first six rows of the seats below. Intensified damage to architectural components such as cladding, ceilings, and partitions was noted in the vicinity of building separation joints. Damage to nonstructural partitions was observed, and seemed more severe in mid-rise steel moment frame structures.

The article attributes great monetary losses to failure or lack of seismic restraints to utility systems. The reconnaissance observers noted failures of fire sprinkler mains and piping joints with the resulting flooding collapsing ceilings and, in some cases, causing the abandonment of entire floors. Damage to ceilings and sprinkler systems temporarily closed a terminal at the San Francisco International Airport. A software company located five miles from the epicenter was forced to evacuate when water pipes ruptured and flooded their buildings.

The high-tech industry plays a critical role in the San Francisco Bay area, and the report includes performance data from 32 data processing facilities. The information gathered includes performance on raised access floors, process equipment, tape and cartridge racks, ceilings, and support utilities. The facilities did not report extensive

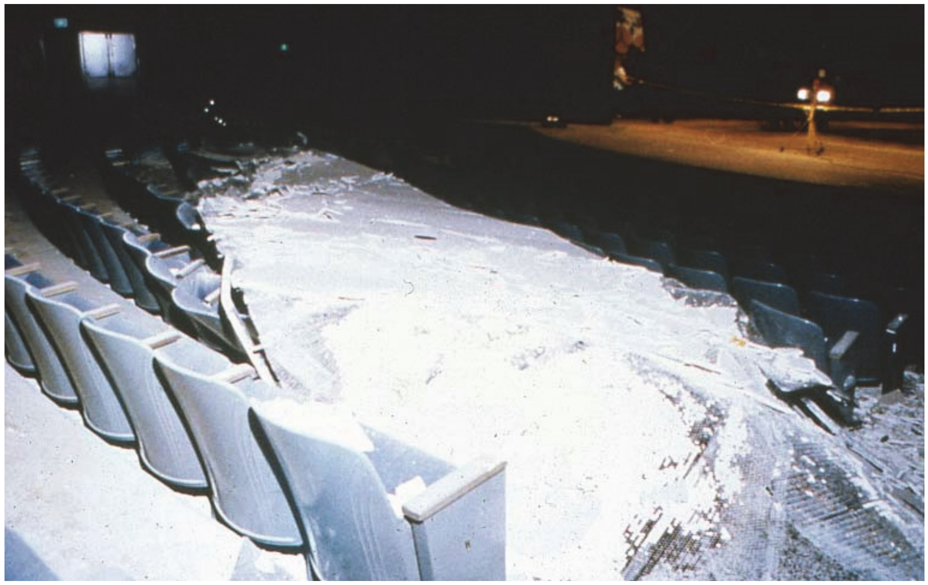


Figure 2-4 Collapsed heavy ceiling in the Geary Theater, San Francisco, California, 1989 Loma Prieta earthquake (NISEE, 2016c).

damage. Four facilities reported minor damage to raised access floors. Only one facility reported that some data processing equipment was anchored. All facilities reported that equipment on casters, some of which displaced up to four feet, had no serious damage. Some tape racks overturned, and tapes dislodged from racks that were braced or anchored to walls. Many pieces of desktop equipment fell to the floor but in most cases only minor repairs were required to restore them to operation. File and storage cabinets fell, which hampered resuming operations. In one case, the items blocked the only exit to a room. Operations were most disrupted by loss of electrical power. Where installed, emergency power systems generally functioned. One case of flooding due to a ruptured six-inch sprinkler line was reported. Items in basements sustained little damage.

In 1975, California adopted seismic provisions for elevators. The code included retroactive requirements that had to be implemented within seven years of the adoption of the seismic provisions. Following the 1989 Loma Prieta earthquake, information on the performance of elevators was collected. Questionnaires were sent to nine major elevator service companies. At the time of the *Earthquake Spectra* article, seven of the companies had responded. Unfortunately, only the number of damaged elevators was reported, and the total number of elevators subject to earthquake shaking is not known. Counterweights came out of the guide rails in a large number of responses, although there are few precise details as to the cause. There were six reported incidents of counterweights striking cars, although required protective devices were intended to prevent this. There were reported inconsistencies in the operation of the protective devices; in a single structure, some devices triggered and others did not. Compared to earlier earthquakes, a higher incidence of damage to hydraulic elevators was reported.

The California Office of Statewide Health Planning and Development (OSHPD), which had jurisdiction over 428 hospitals and healthcare facilities in the impacted region, collected more comprehensive data on elevator performance. The *Earthquake Spectra* article contains the results of the OSHPD survey, as well as a case study of the performance of elevators at the Stanford University Hospital, a facility with 41 elevators.

2.4.2 Correlation between Recorded Building Data and Non-Structural Damage

The 1989 Loma Prieta earthquake triggered strong motion records at 131 instrument sites, including an instrumented thirteen-story building in San Jose, California. This building, the Santa Clara County Government Center, was the subject of a study by S. Rihal to correlate the California Strong Motion Instrumentation Program (CSMIP) response data with observed nonstructural component damage. The CSMIP/94-04 report, *Correlation Between Recorded Building Data and Non-Structural Damage During the 1989 Loma Prieta Earthquake* (Rihal, 1994), was published with the findings of this study.

The building was designed in 1972 and features a complete steel moment resisting frame. Sensors were located at grade, the second, seventh, and twelfth floors, and at the roof. The peak ground acceleration was about 0.10g, and the peak acceleration at the roof was 0.34g. Although the peak ground accelerations were low, the building suffered substantial damage to contents on the seventh, ninth, tenth, and eleventh floors, and some minor nonstructural damage at the sixth and eighth floors. The dynamic behavior of the Santa Clara County Government Center is somewhat unusual, because very little damping was provided by nonstructural components. The exterior shell of the building is a glazed curtain wall, and there are almost no full-height partitions. As a result, the roof the building experienced nearly 48 cycles of motion in which the peak floor acceleration exceeded 0.05g and persisted for over 100 seconds. The large number of cycles contributed to the high levels of content damage.

Collection of nonstructural damage data is challenging because repairs are often undertaken almost immediately following an earthquake. In this instance, a video was made documenting nonstructural damage shortly after the earthquake, which allowed for classification of damage patterns and failure modes. Different failure modes for cabinets were identified, and damaged items were identified by floor, including estimates of the percentage of different items that were damaged and whether they tipped, broke, or displaced.

The report illustrates the possibilities with regard to collection of nonstructural component performance data if an effort is made to collect data immediately following an earthquake.

2.5 January 17, 1994 Northridge Earthquake

On January 17, 1994, a strong earthquake struck the San Fernando Valley west-northwest of downtown Los Angeles. The magnitude-6.7 event struck the same general region that was impacted by the 1971 San Fernando earthquake. Two studies prepared after this earthquake are summarized. The first covers a series of reports on nonstructural design forces, water damage, and elevator performance produced for OSHPD. The second is a report on performance of fire sprinkler systems.

2.5.1 OSHPD Studies

Following the Northridge earthquake, OSHPD funded a series of studies on nonstructural performance. The report, *1994 Northridge Hospital Damage: OSHPD Studies: Water, Elevator, Nonstructural* (OSHPD, 1996), provides details about water damage, elevator performance, and seismic design factors for nonstructural components in hospitals. These studies utilized extensive field reports generated by OSHPD teams immediately following the earthquake.

Seismic Design Factor Study

The Seismic Design Factor study focused on data obtained from the Olive View Medical Center, which included the main building, an instrumented six-story steel plate shear wall structure, and the single-story central plant. The Medical Center was built on the site of the former Olive View Hospital, which was demolished following the 1971 San Fernando earthquake. The replacement buildings were designed in 1976 and completed in the mid-1980s. The medical center site is located ten miles from the epicenter of the Northridge earthquake. The main building experienced very strong shaking, with a peak horizontal acceleration of 0.8g measured at the ground floor and 1.5g measured at the roof. The authors of the OSHPD study had access to the original drawings and calculations for the nonstructural components and systems.

Detailed data was collected on approximately one hundred mechanical and electrical components in the main building, and twenty components in the central plant. Information is provided for the component dimensions and weight, the number and arrangement of anchors, location of the component in the building, and whether the component was damaged. Also include are estimates of the design lateral force on the structure, and, in some cases, drawings indicating the installation of components. Equipment was designed for a horizontal allowable strength design force of either 50% or 75% of the operating weight, depending on whether the component function was deemed critical. Components were designed for vertical forces equal to one-third the horizontal force. The design drawings for equipment anchorage in the central plant specified a 100% increase in design force for flexible or flexibly-mounted equipment.

The design demand-to-capacity ratios for the equipment anchors were estimated. For some components, a comparison between the design force and the actual demand was made. The observed demands were also compared to then current standards: the 1994 *Uniform Building Code* (ICBO, 1994), the *1995 California Building Code* (CBSC, 1995), and the 1994 *NEHRP Recommended Seismic Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1995).

Given the strong shaking experienced at the site, there was surprisingly little damage to equipment. Of the 221 televisions installed in the main building, 97 were damaged. About half of the damage to the televisions was caused by failures of the wall brackets. Damage to ceiling systems and piping was extensive, forcing the evacuation of approximately 300 patients.

Water Damage Study

The Water Damage Study included thirteen acute care hospitals in the Los Angeles area. The report includes qualitative summaries of damage to equipment and systems, sometimes floor-by-floor, and some quantitative data on sprinkler and pipe failures. Water damage was a critical factor in the decision to curtail operation or evacuate some hospitals. While displacement of roof mounted tanks that sheared piping connections occurred in some buildings, the report identified failures in heating-system hot water line connections to unbraced duct-mounted reheat coils and damage to sprinkler systems as the primary sources of water damage.

For each of the facilities, a description of the nature and extent of water damage is provided, along with the probable cause. Information on both the physical damage to piping and equipment as well as the response of the facility staff was considered. It was observed that operating engineers sometimes had difficulty turning off pumps to stop the flow of water, and that inability of the valves to close tightly extended the length of time that water flowed under pressure, increasing the area of water damage. Cast iron components such as valves and flanges were often the weakest link in the systems. As shown in Figure 2-5, fire sprinkler components failed when differential movements occurred at hard ceilings and walls, where heads struck other building components, and where lines crossed seismic separations. Bracing failures (specifically one-sided C-clamps used for gravity hangers) contributed to the pipe movement and failures at screwed joints.

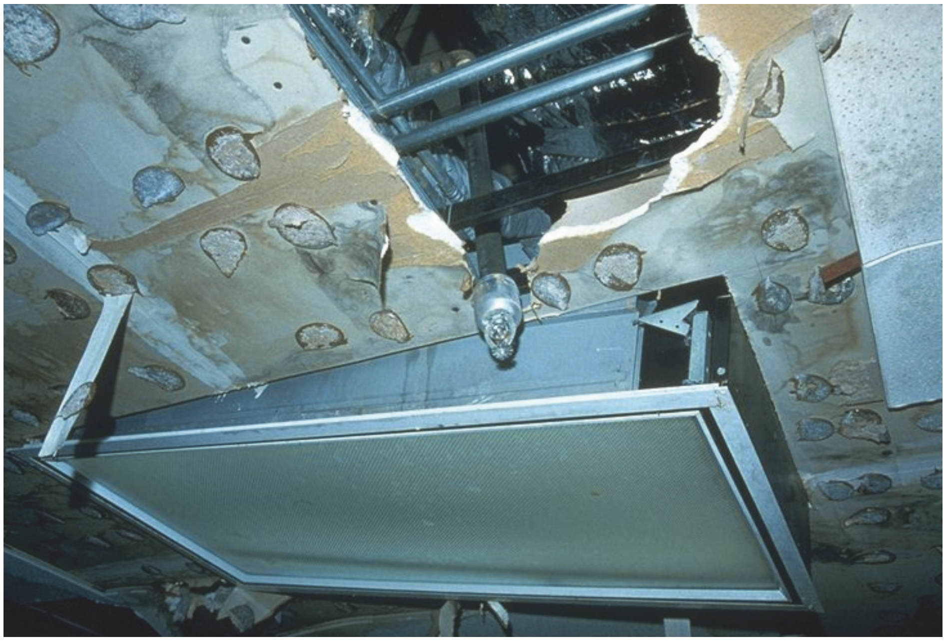


Figure 2-5 Fractured sprinkler line and dislodged light fixture in the Olive View Medical Center, 1994 Northridge earthquake (NISEE, 2016d).

Elevator Study

The Elevator Study included nine acute care hospitals in the Los Angeles area, with a total of 100 traction and 12 hydraulic elevators. As part of the study, the elevators were surveyed and personnel were interviewed. Damage to elevators was fairly widespread, despite legislation requiring seismic upgrades enacted in the mid-1970s.

A variety of failure modes were identified in the report. Issues with counterweight rail distortion, bracket failures, and frame distortions were common, as were reports of bent car stabilizers. Traction elevator ropes and cables snagged or tangled, and some anchorage failures of elevator machinery were noted. Only one of the twelve hydraulic elevators had equipment problems. There were a number of reports of damage to the hoistway walls, with some damage due to impact of counterweights which came out of their guide rails, and cases where the ceiling panels in the cars fell. The seismic protection devices operated correctly.

In addition to issues with the elevators themselves, elevator operations were hampered by failure of emergency power systems and lack of emergency lighting in machine rooms. Lack of wiring diagrams for the equipment also hampered recovery efforts.

2.5.2 Performance of Fire Sprinkler Systems

Damage to fire sprinkler systems was a significant contributor to losses in the Northridge earthquake. In response, NIST commissioned a study to review the performance of fire sprinkler systems in the earthquake, and recommend steps that could be taken to improve seismic performance. NIST GCR-98-736, *Analysis of Sprinkler Systems Performance in the Northridge Earthquake* (Fleming, 1998), detailed the findings of the study and includes observations on the performance of different components in a fire sprinkler system, such as sway bracing, fasteners to the structure, hangers, and interactions between sprinklers and ceilings. The report also includes a discussion on the evolution of seismic requirements for sprinkler systems, observations on overall system performance in the Northridge earthquake, and suggestions for possible changes to sprinkler design procedures.

One of the challenges faced when investigating the performance of nonstructural components is determining, for a given component installation, the design and construction requirements in force at the time of construction. This is especially true in the case of fire sprinkler systems, where seismic provisions have been included in national installation rules since 1947. Few buildings in the region impacted by the 1994 Northridge earthquake were designed to the latest edition of installation standards, and two-thirds of the installations where damage was reported were installed prior to 1976.

The report cites the City of Los Angeles Department of Water and Power data identifying about 14,000 fire sprinkler systems in their service area at the time of the earthquake, of which 3,300 were in the San Fernando Valley, which was the region most impacted by the earthquake. A survey to obtain information on sprinkler system performance was conducted by the Fire Sprinkler Advisory Board of Southern California. Only 225 responses were received from the 2,000 survey forms distributed. The survey did not ask for the year of installation for the system. Still, the nature and pattern of observed failures could be determined from the responses received, and a list of the different types of damage and the probable causes is included in the report. Fastener failures were noted, notably power-driven fasteners. Beam clamps without retainer bars and screw fasteners in wood were also identified as contributors to damage. Interactions between ceilings and sprinkler heads were noted, as were failures at piping joints, especially at threaded fittings. There were failures of pipe hangers, and interaction between sprinkler piping and other structural and nonstructural components. The study also summarizes damage data collected in the OSHPD Water Damage Study. Interactions between ceilings and sprinkler heads and failures of threaded connections at the top of sprinkler drops were reported.

This report provides a list of performance concerns. For each concern, a short summary of the development of current sprinkler requirements related to the concern, a summary of performance observations in the 1994 Northridge earthquake relative to those concerns, and possible changes to sprinkler requirements to be considered in the future are provided. Many of the recommendations were incorporated into later editions of the standard for installation of sprinkler systems.

2.6 February 27, 2010 Chile Earthquake

On February 27, 2010, a very strong magnitude-8.8 earthquake occurred off the coast of Maule, Chile causing heavy damage across a large region. A paper, prepared by Miranda et al. (2012), described the performance of nonstructural components and appeared in a special edition of *Earthquake Spectra* devoted to the 2010 Chile earthquake. Much of the data presented in this paper were gathered on a reconnaissance mission to some of the major cities impacted by the earthquake.

Although the Chilean seismic code NCh 433.Of96, *Earthquake Resistant Design of Buildings* (INN, 1996), contains provisions for nonstructural components, they are not often enforced. The Chilean code generally requires higher design forces for nonstructural components than those in the United States. Drift requirements for the two countries are comparable.

The new terminal building at the international airport in Santiago, completed in 1994, suffered minor structural damage, but sustained crippling nonstructural damage. More than 80% of ceilings were damaged, and fire sprinkler system failures resulted in extensive water damage. Most other nonstructural systems and equipment in the terminal suffered heavy damage.

Hospitals in the region also suffered, with 63% sustaining nonstructural damage requiring repairs. Damage to elevators and contents was common, although no damage to medical equipment such as imaging equipment or surgical lights was observed. No damage due to flooding was observed, although only the most modern hospitals in Chile have fire sprinkler systems.

Nonstructural damage was common in all types of commercial buildings, although most buildings had excellent structural performance. Damage to ceilings and glazing was evident, as was partition damage and damage to mechanical components. Nonstructural damage to the University of Concepción Marine Biology Station was severe, with the laboratory components and equipment completely destroyed.

In general, U.S.-style suspended acoustical ceilings were extensively damaged with a large percentage of the ceiling panels falling. Ceiling damaged increased in the upper stories of buildings. Few buildings were equipped with fire sprinklers, although about half of the buildings with sprinklers that were inspected had water leakage. Sprinkler bracing practices appeared to be similar to those in the United States, and brace anchorage failures were observed.

About half of the elevators in the area affected by the earthquake were damaged. Damage included counterweight and cab derailments, guide rail damage, jammed doors, and shifted equipment. Engineered curtain walls and façade elements performed well, especially in buildings constructed after 1985.

2.7 August 24, 2014 South Napa Earthquake

On August 24, 2014, a magnitude-6.0 earthquake struck northern California with the epicenter about six miles from the city of Napa. Following the earthquake, FEMA commissioned an effort to collect data on the performance of structures and nonstructural components in the impacted area, with a focus on collection of data for all buildings within a 1,000-foot radius of the strong motion recording device at Station N016 of the Northern California Seismic Network operated by the U.S. Geological Survey. The peak horizontal ground acceleration recorded at this station was 0.65g. The results of this study were reported in FEMA P-1024, *Performance of Buildings and Nonstructural Components in the 2014 South Napa Earthquake* (FEMA, 2015a). Buildings constructed in accordance with current codes performed well structurally, although older structures with known vulnerabilities such as poor wall-to-roof connections suffered some damage.

The majority of losses in the South Napa earthquake were the result of nonstructural damage. Of the 68 buildings within the survey area around Station N016, 22 were reported to have damage to at least 5% of the exterior glazing, and four buildings experienced damage to at least 50% of the glazing. In some storefronts, the glass did not crack or shatter, but the gaskets holding the glass in the frames loosened, allowing the panes to shift. Some modern structures suffered substantial damage to light-frame curtainwalls with loosening or loss of adhered veneer, as shown in Figure 2-6. The exterior stud and stucco wall of one moment frame structure was not detailed to accommodate story drift and during the earthquake one wall detached from the floor diaphragm. Partition damage due to shaking was generally very light, although there was substantial damage to partitions in buildings that suffered water damage. The most commonly observed form of damage to suspended acoustic tile ceilings was fallen tiles. Failure of splices, as well as damage at the "fixed" and "free" ends of the ceiling grid was observed in several buildings.

Some unanchored equipment shifted or overturned. Failures of anchored equipment occurred in older installations which would not comply with current codes. Rooftop equipment was more heavily damaged than equipment located elsewhere in buildings. Pendant light fixtures in some buildings fell. In one building, nearly all vibration isolated components had anchorage failures, often associated with splitting of unreinforced housekeeping slabs.

Failures of sprinkler piping were the cause of substantial damage, in some cases flooding entire floors. Causes of the failures include impact of heads with other piping, nonstructural components, or structural members. Failures at threaded pipe fittings were also noted. Flooding was aggravated by the fact that, in most areas, water service was not lost. The earthquake occurred in the early morning hours, and occupants had trouble getting to their businesses to turn off the water supply.

Figure 2-6 Adhered veneer damaged during the 2014 South Napa earthquake (FEMA, 2015a).

Virtually all buildings in the area suffered some contents damage, and a number of wineries in the area suffered losses when stacks of wine barrels on portable barrel racks collapsed. Two facilities suffered significant structural damage due to the impact of toppling stacks of wine barrels.

2.8 Observations on Post-Earthquake Damage Assessments

Fifty years after the patterns and causes of nonstructural damage were documented following the 1964 Great Alaska earthquake, similar damage patterns are found in many earthquakes occurring more recently. This is due in part to the age and diversity of the building stock, but is also due to a persistent reluctance to deal with critical aspects of the nonstructural problem: design responsibility, code enforcement, and construction oversight. These issues were identified in the Ayers et al. (1967) report on the 1964 Great Alaska earthquake, and they continue today. Although building codes have made substantial advances in nonstructural seismic design, translating this knowledge consistently into finished construction is an ongoing challenge. Technical enhancements, such as changes to the elevator design requirements in California following the 1971 San Fernando earthquake, do not always lead to improved performance. Earthquakes in California over the following 25 years have revealed continued vulnerabilities.

The 1994 Northridge earthquake resulted in substantial changes in the seismic design of nonstructural components. The nonstructural design force equations, anchorage to concrete, the design of steel storage racks, the need for seismic qualification of essential equipment, and the design of fire sprinkler systems were all examined, and many changes were made to incorporate lessons learned. The effectiveness of the latest codes and standards has yet to be tested in a strong earthquake.

In some ways, the focus and detail achieved in the 1967 Ayers report on the 1964 Great Alaska earthquake has not been equaled since. Information on nonstructural damage is perishable unless a building has substantial structural damage that prevents immediate clean-up and repairs. If clean-up and repairs occur quickly, critical information on the source and effects of nonstructural failures is lost. It also seems increasingly more difficult now to obtain reliable damage information, as building owners, engineers, and contractors become increasingly reluctant to share the information they have. There is little incentive to report nonstructural performance, good or bad, following an earthquake. When describing nonstructural damage, most recent earthquake reconnaissance reports do not indicate the building type and age, the location of the component in the structure, or an opinion as to whether the component was installed to the applicable code or standard. This presents a serious challenge to assessing the effectiveness of design practices for nonstructural components and systems based on observed performance in past earthquakes.

Chapter 3

Seismic Design and Evaluation Methodologies for Nonstructural Components

3.1 Development of Seismic Design Provisions for Nonstructural Components in the UBC and *NEHRP Provisions*

Building code provisions for seismic design of nonstructural components were first incorporated in the very first edition of the *Uniform Building Code* (ICBO, 1927). Stimulated by lessons learned in earthquakes over the next nine decades, the building codes have incorporated lessons learned in damaging earthquakes and, more recently, from full-scale shake table tests. In this section, a summary of the evolution of nonstructural seismic design requirements in the *Uniform Building Code, International Building Code,* and *National Earthquake Hazards Reduction Program (NEHRP) Provisions* is presented. More recently, ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010), has become the reference standard for nonstructural structural design in the United States. ASCE/SEI 7-10 is adopted by model building codes such as the 2015 *International Building Code* (ICC, 2015) and the *2016 California Building Code* (CBSC, 2016).

The seismic design requirements for nonstructural elements and the expected changes in ASCE/SEI 7-16 Standard, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2016) are discussed in Section 3.2. Sample force calculations are provided to illustrate the application of the seismic design force equation for a cantilevered rooftop parapet and equipment mounted on the roof of the same building. A summary of the changes in the design force levels for nonstructural components from 1927 to the present are discussed in Section 3.3. Recent changes in the calculation of the seismic design force for anchorage of nonstructural components to concrete and masonry are presented in Section 3.4. In Section 3.5, the history of nonstructural component anchorage to concrete is discussed. In Section 3.6, international nonstructural design provisions from outside of the United States are presented for comparison to the current design philosophy in the United States. Issues surrounding the enforcement of seismic requirements for nonstructural components are presented in Section 3.7. Finally, evaluation of seismic resistance of existing nonstructural components and anchorage is discussed in Section 3.8.

3.1.1 Early Provisions 1927-1976

From 1927 to 1976, slow progress was made on seismic design provisions for nonstructural components. Towards the end of this period, the nature of building construction began to change rapidly with the introduction of modern heating and air condition systems, complex electrical systems, and the modern cladding systems. Seismic lateral-force resisting systems for buildings also changed and became more flexible with the shift twoards drift-controlled moment frames over stiff braced and shear wall systems. In addition, damaging earthquakes, even at levels below what would be considered a design event, increased interest in improving the seismic performance of nonstructural components and systems.

1927 UBC

The first edition of the *Uniform Building Code* (UBC) was published in 1927 (ICBO, 1927). Chapter 23 contained requirements for dead, live, and wind loads. Seismic provisions were located in an appendix of the UBC. Section 2311 began with the statement that "Every building and every portion thereof, except Type IV and V buildings and all one-story buildings which are less than twenty feet in height shall be designed and constructed with bracing to resist the stresses produced by lateral forces as provided in this section." Type IV and V buildings are those of metal frame and wood frame construction, respectively. When the allowable soil bearing pressure for the building site is 4,000 psf or more, the horizontal forces are 7.5% of the total dead load plus live load of the building above the plane of interest. When the allowable soil bearing pressure for the building site is below 4,000 psf, horizontal forces are increased to 10% of the total dead load plus live load of the building above the plane of interest. There are no specific requirements for mechanical, electrical, and plumbing (MEP) components, but the "every portion thereof" clause could be interpreted to apply to the design of nonstructural components. Applying this approach to determine the out-of-plane seismic design force on a wall would result in either 7.5% or 10% of the the wall weight, dependent on the allowable soil bearing pressure at the site. Since in 1927, the seismic design force is calculated at the allowable stress design level, a load and resistance factor design (LRFD) or, also known as, strength design level could be converted to a strength level by using the now generally accepted adjustment factor of 1.4, resulting in either 10.5% or 14% of the wall weight.

1935 UBC through 1958 UBC

Seismic design provisions remained in an appendix of the 1935 UBC (ICBO, 1935). The seismic design provisions apply to all buildings except those of Type V (wood frame) construction with Group I occupancy (residential dwellings) and shorter than 25 feet in height. The 1935 UBC includes an explicit equation and a table of values for seismic design of nonstructural components. Values are provided for bearing walls and curtain walls; parapets; ornamentation and appendages; and towers, tanks, chimneys, and penthouses. The designation of "ASD" is added to F_p in this report when the design force is calculated at the allowable stress design (ASD) level. The "-ASD" portion of the subscript is not used when the design is calculated at the strength level used in conjunction with a load and resistance factor design (LRFD) philosophy. Provisions for the seismic design of nonstructural components are unchanged in the 1937, 1940, 1943, 1946, 1952, 1955, and 1958 editions of the UBC (ICBO, 1937; 1940; 1943; 1946; 1952; 1955; 1958). The force equation for nonstructural seismic design used in the 1935 UBC through the 1958 UBC is:

$$F_{p\text{-}ASD} = CW_p \tag{3-1}$$

where:

$F_{p\text{-}ASD}$ = lateral seismic force at the allowable stress design (ASD) level;

C = horizontal seismic force factor and ranges from 0.05 for walls and glazing to 0.25 for cantilevers and appendages for building sites located in Zone 1, and C either doubles for Zone 2 or quadruples for Zone 3; and

W_p = weight of component.

The introduction of C marks the first attempt in the UBC to vary the lateral seismic force based on different types of nonstructural components.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco located in Seismic Zone 3 (Coastal California) is:

$$F_{p\text{-}ASD} = CW_p = (0.25 \times 4)W_p = 1.0W_p \tag{3-2}$$

Conversion of the design force to LRFD level from ASD level yields:

$$F_p = 1.4F_{p\text{-}ASD} = 1.4 \times 1.0W_p = 1.4W_p \tag{3-3}$$

1961 UBC and 1964 UBC

In the 1961 UBC (ICBO, 1961), seismic design provisions were moved from an appendix and brought directly into Chapter 23 that contained the provisions for dead, live, and wind loads. The coefficient, C, which accounts for differences in seismic risk and component behavior in earlier editions of the code, was replaced by separate coefficients to account for the Seismic Zone (Z) and the type of component (C_p). Requirements for the seismic force-resisting system were significantly changed and are more specific in nature. The exemption for Type V buildings less than 25 feet in height was eliminated. The equation and table for seismic design of nonstructural components were changed. The seismic design provisions for nonstructural components are unchanged in the 1964 UBC (ICBO, 1964). The force equation for nonstructural seismic design used in the 1961 UBC and the 1964 UBC is:

$$F_{p\text{-}ASD} = ZC_pW_p \tag{3-4}$$

where:

$F_{p\text{-}ASD}$ = lateral seismic force at the allowable stress design (ASD) level;

Z = Seismic Zone factor equal to 0.25 for Zone 1, 0.5 for Zone 2; and 1.0 for Zone 3;

C_p = horizontal force factor that ranges from 0.2 to 2.0, dependent on the type of component being designed; and

W_p = weight of component.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco located in Seismic Zone 4 (Coastal California) is:

$$F_{p\text{-}ASD} = ZC_pW_p = 1 \times 1.0W_p = 1.0W_p \tag{3-5}$$

Conversion of the design force to LRFD level from ASD level yields:

$$F_p = 1.4F_{p\text{-}ASD} = 1.4 \times 1.0W_p = 1.4W_p \tag{3-6}$$

1967 UBC and 1970 UBC

In the 1967 UBC (ICBO, 1967), connections for exterior panels were added to the list of components. The forces for towers, tanks, towers and plus contents, chimneys, smokestacks, and penthouses in slender structures with a height to width ratio greater than 5 were increased by 50%. Otherwise, nonstructural seismic design provisions were unchanged. No changes to the nonstructural seismic design provisions were made in the 1970 UBC (ICBO, 1970).

1973 UBC

In the 1973 UBC (ICBO, 1973), suspended ceiling framing and storage racks over 6 feet in height plus contents were added to the list of components. Otherwise, seismic design provisions for nonstructural components are unchanged.

1976 UBC

The 1976 UBC (ICBO, 1976) contains one force equation for nonstructural component lateral design. Requirements for equipment and machinery were explicitly added for the first time, with special requirements for flexible and flexibly mounted equipment and machinery. In addition, factors to account for importance of the structure (*I*) and soil-structure resonance (*S*) were added to the equation. When the value of C_p exceeded 1.0, the value of S and I need not exceed 1.0. The requirements for suspended ceilings and storage racks, were expanded. The equation for nonstructural seismic design of components, and equipment and machinery anchorage used in the 1976 UBC is:

$$F_{p\text{-}ASD} = ZIC_pSW_p \tag{3-7}$$

where:

$F_{p\text{-}ASD}$ = lateral seismic force at the allowable stress design (ASD) level;

Z = Seismic Zone factor equal to 0.1875 for Zone 1, 0.375 for Zone 2, 0.75 for Zone 3; and 1.0 for Zone 4;

I = importance factor based on Occupancy Category equal to 1.5 for essential facilities, 1.25 for assembly use (over 300 people in one room), and 1.0 for all other occupancies;

C_p = horizontal force factor that ranges from 0.3 to 1.0, dependent on the type of component design designed (increased values are used if the equipment serves a life safety function);

S = site-structure resonance factor calculated as $(1.0 + T / T_s - 0.5(T / T_s)^2)$ when $T / T_s \leq 1.0$) or $(1.2 + 0.6(T / T_s) - 0.3(T / T_s)^2)$ when $T / T_s > 1.0$) and is never taken to be less than unity;

T = fundamental period of the building;

T_s = characteristic site period; and

W_p = weight of component.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco located in Seismic Zone 4 (Coastal California) is:

$$F_{p\text{-}ASD} = ZIC_pSW_p = 1 \times 1.0 \times 1.0 \times 1.0W_p = 1.0W_p \tag{3-8}$$

Conversion of the design force to LRFD level from ASD level would yield:

$$F_p = 1.4F_{p\text{-}ASD} = 1.4 \times 1.0W_p = 1.4W_p \tag{3-9}$$

For this example, the value of S was taken to be 1.0, since the value of C_p was 1.0 and the product of these two values need not exceed 1.0.

The design force for an air-handling unit with rigid anchorage on the roof of the same building in downtown San Francisco located in Seismic Zone 4 is:

$$F_{p\text{-}ASD} = ZIC_pSW_p = 1 \times 1.0(0.2 \times 1.5) \times 1.5W_p = 0.45W_p \tag{3-10}$$

Without T_s established by geotechnical studies, the code allows a value of 1.5 to be assumed for S, and this assumed value was used in this calculation. Also, it was assumed that the ratio of roof height to building plan dimension in the direction of the earthquake was larger than 5, and therefore the value of C_p, equal to 0.2 for non-essential mechanical equipment, was increased by a factor 1.5. If the ratio was 5 or less, the design force is only 0.30 W_p.

Conversion of the design force to LRFD level from ASD level would yield:

$$F_p = 1.4F_{p\text{-}ASD} = 1.4 \times 0.45W_p = 0.63W_p \qquad (3\text{-}11)$$

Requirements were added for story drift and building separations. Lateral deflections or drift of a story relative to its adjacent stories were limited to 0.005 times the story height unless it was demonstrated that greater drift can be tolerated. All portions of structures were to be designed and constructed to act as an integral unit in resisting horizontal forces unless separated structurally by a distance sufficient to avoid contact under deflection from seismic action or wind forces. For nonstructural precast wall panels, the allowance for relative movements between panels was increased from 2 to (3.0 / K) times the story drift caused by seismic forces. The horizontal force factor, K, ranged from 1.33 for bearing wall box systems to 0.67 for ductile moment resisting space frames resisting all of the seismic lateral forces.

Requirements were added for essential facilities which must be safe and usable after an earthquake, such as hospitals, fire stations, and emergency operation centers. Design and detailing of equipment which must remain in place and be functional following a major earthquake was required. Nonstructural components were design for not less than (2.0 / K) times the story drift caused by required seismic forces nor less than the story drift caused by wind.

3.1.2 Parallel Developments 1979-2002

The late 1970s saw rapid advances in the seismic design of nonstructural components, beginning with the publication of ATC-3-06, *Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC, 1978), which profoundly influenced the development of seismic design requirements.

ATC-3-06

ATC-3-06 introduced a number of concepts that changed the way engineers approach seismic design of nonstructural components. ATC-3-06 provides far more extensive provisions for nonstructural components than found in the UBC, along with commentary explaining the rationale for these new requirements. Nonstructural requirements are based on Performance Characteristic Levels, related to type of component and seismic hazard level. There are four Performance Characteristic Levels: Superior; Good; Low; and NR (Not Required). A performance criteria factor, P, was assigned to each Performance Characteristic Level, functioning in the same manner as the importance factor, I, is used in modern building codes and load standards. The performance criteria factor, P, applies to an individual component rather than the structure as a whole. The seismic design force calculation for mechanical and electrical components considers amplification of lateral loads due to dynamic interaction between the component and the structure. It also captures expected floor acceleration increases in the upper levels of the structure.

Components with Performance Characteristic Levels of Superior or Good requires certification from the manufacturer that component will not sustain damage when subjected to the seismic design force, and special inspection is required. Periodic special inspection is also required for some architectural, mechanical and electrical components with Performance Characteristic Levels of Superior or Good.

The rationale behind the ATC-3-06 nonstructural design provisions is detailed in 28 pages of commentary. Performance Levels for nonstructural components are introduced based on the function of the building and the hazard presented by the components. Design goals are given for architectural, mechanical, and electrical components by performance level. For example, for a Performance Level of Superior, the design goals for mechanical and electrical component are: (1) high resistance to static and dynamic forces; (2) operating functions unimpaired; (3) no broken piping regardless of size; and (4) no interruptions of utility services other than the normal time to transfer functions to alternate sources. A performance matrix is provided for each nonstructural component, with recommend Performance Characteristic Levels for different building occupancies. However, nearly two decades were needed for these concepts to be incorporated in model building codes.

1979 UBC

The 1979 UBC (ICBO, 1979) contains one force equation for nonstructural component lateral design. The S factor, having a default value of 1.5, in the 1976 UBC was removed from the force equation. In the 1979 UBC, most values of the C_p factor were increased by 50%, so there was no effect on the design forces for most components. The equation for nonstructural seismic design of components, and equipment and machinery anchorage used in the 1979 UBC is:

$$F_{p\text{-}ASD} = ZIC_pW_p \tag{3-12}$$

where:

$F_{p\text{-}ASD}$ = lateral seismic force at the allowable stress design (ASD) level;

Z = Seismic Zone factor equal to 0.1875 for Zone 1, 0.375 for Zone 2, 0.75 for Zone 3; and 1.0 for Zone 4;

I = importance factor based on Occupancy Category equal to 1.5 for essential facilities, 1.25 for assembly use (over 300 people in one room), 1.0 for all other occupancies, and 1.5 for components required for life safety or containing hazardous materials;

C_p = horizontal force factor that ranges from 0.3 to 0.8 dependent on the type of component being designed; and

W_p = weight of component.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco located in Seismic Zone 4 (Coastal California) is:

$$F_{p\text{-}ASD} = ZIC_pW_p = 1 \times 1.0 \times 0.8W_p = 0.80W_p \tag{3-13}$$

Conversion of the design force to LRFD force level from ASD force level would yield:

$$F_p = 1.4F_{p\text{-ASD}} = 1.4 \times 0.80W_p = 1.12W_p \tag{3-14}$$

The design force for an air-handling unit with rigid anchorage on the roof of the same building in downtown San Francisco located in Seismic Zone 4 is:

$$F_{p\text{-}ASD} = ZIC_pW_p = 1 \times 1.0 \times 0.3W_p = 0.30W_p \tag{3-15}$$

Conversion of the design force to LRFD level from ASD level would yield:

$$F_p = 1.4F_{p\text{-ASD}} = 1.4 \times 0.30W_p = 0.42W_p \tag{3-16}$$

The footnote of Table 23-J of the 1979 UBC mentions that C_p for flexible or flexibly-anchored equipment "shall be determined with consideration given to both the dynamic properties of the equipment and machinery and to the building or structure in which it is placed but not less than the listed values." The code implies that the values listed in the table are for rigidly-anchored components. However, the code does not provide guidelines how to establish C_p values for flexible or flexibly-anchored equipment or what to assume when C_p cannot be established by analyses. Therefore, the design force established using Eq. 3-15 should be considered the minimum allowed by the 1979 UBC.

1982 UBC and 1985 UBC

The 1982 UBC (ICBO, 1982) nonstructural seismic design provisions are essentially the same as the 1979 UBC. Nonstructural seismic design provisions were unchanged in the 1985 UBC (ICBO, 1985) except that fire sprinklers, supports and bracing for equipment racks and piping for hazardous materials, and raised access floors were explicitly added to the requirements.

This publication is available free of charge from: https://doi.org/10.6028/NIST.GCR.17-917-44

1985 NEHRP Provisions

The nonstructural requirements of the 1985 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings, Part 1: Provisions* (FEMA, 1985) were identical to those in the update of ATC-3-06, *Amended: Tentative Provisions for the Development of Seismic Regulations for Buildings* (ATC, 1984), a revision to the original 1978 edition of ATC-3-06. Seismic design of nonstructural components is covered in Chapter 8 of the 1985 *NEHRP Provisions*. Requirements for mechanical and electrical components have a slightly different equation than the equation for architectural components. Design forces are calculated at the strength design (LRFD) level.

For architectural components, the equation is:

$$F_p = A_v C_c P W_c \tag{3-17}$$

where:

F_p = lateral seismic force at the strength design (LRFD) level;

A_v = effective peak velocity-related acceleration factor that ranges from 0.05 to 0.40, dependent on the seismicity of the site;

C_c = horizontal force factor that ranges from 0.6 to 3.0, dependent on the type of architectural component being designed;

P = performance criteria factor that ranges from 0.5 to 1.5, dependent on the Seismic Hazard Exposure Group (Group III is for essential facilities, Group II is for assembly buildings, larger schools, jails and select other occupancies, and Group I is for all other buildings, including office buildings); and

W_c = weight of the architectural component.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco located located on site with $A_v = 0.4$ is:

$$F_p = A_v C_c P W_c = 0.4 \times 0.9 \times 1.5 W_c = 0.54 W_c \tag{3-18}$$

Note that the 1985 *NEHRP Provisions* do not have a separate category for parapets or cantilevers. The commentary noted that the authors of the 1985 *NEHRP Provisions* "could not justify a difference between a parapet and a cantilever portion of an exterior wall. The poor history of unreinforced masonry parapets, which was the basis of prior high C_c values, should not be transferred to newer and properly designed systems." As a result, the $C_c = 0.9$ value for exterior nonbearing walls was used along with a performance criteria factor of 1.5 considering that poor seismic performance of a parapet could have life safety consequences.

For mechanical and electrical components, the equation is:

$$F_p = A_v C_c P a_c a_x W_c \tag{3-19}$$

where:

F_p = lateral seismic force at the strength design (LRFD) level;

A_v = effective peak velocity-related acceleration factor that ranges from 0.05 to 0.40, dependent on the seismicity of the site;

C_c = horizontal force factor that ranges from 0.67 to 2.0, dependent on the type of mechanical or electrical component component being designed;

P = performance criteria factor and ranges from 0.5 to 1.5 dependent on the Seismic Hazard Exposure Group (Group III is for essential facilities; Group II is for assembly buildings, larger schools, jails and select other occupancies; and Group I is for all other buildings, including office buildings);

a_c = attachment amplification factor related to the component supporting mechanism and taken as 1.0 for fixed (direct) connection, 1.0 for a seismic activated restraining device connection, 1.0 for a resilient support system where $(T_c / T) < 0.6$ or $(T_c / T) > 1.4$, or 2.0 for $0.6 \leq (T_c / T) \leq 1.4$;

T = fundamental period of the building;

T_c = fundamental period of the component;

a_x = amplification factor determined by $1 + (h_x / h_n)$;

h_x = component elevation in structure relative to grade;

h_n = height of the building; and

W_c = weight of mechanical or electrical component.

The design force for an air-handling unit with direct connection ($a_c = 1.0$) to the roof ($a_x = 2.0$) of the same building in downtown San Francisco is:

$$F_p = A_v C_c P a_c a_x W_c = 0.4 \times 2.0 \times 0.5 \times 1.0 \times 2 W_c = 0.80 W_c \tag{3-20}$$

Note that for such a case, a Performance Characteristic Level of “Low” was allowed per Table 8-C of the 1985 *NEHRP Provisions*, and the value of P is 0.5.

1988 UBC

The 1988 UBC (ICBO, 1988) continued the general design approach for nonstructural components in earlier editions of the UBC, with one equation for nonstructural design. The description of the elements of a nonstructural components requiring design was expanded to include attachment, anchors, and bracing. The use of friction for resisting nonstructural components was restricted. Nonrigid equipment for essential facilities, emergency operations or for life-safety systems, or whose failure would cause a life hazard required design for seismic forces. The list of components was expanded further from those in the 1985 UBC. Equipment weighing less than 400 pounds, furniture, or temporary or movable equipment was explicitly exempted from nonstructural seismic design. The equation for nonstructural design of components, and equipment and machinery anchorage used in the 1988 UBC is:

$$F_{p\text{-}ASD} = ZIC_pW_p \tag{3-21}$$

where:

$F_{p\text{-}ASD}$ = lateral seismic force at the allowable stress design (ASD) level;

Z = Seismic Zone factor equal to 0.075 for Zone 1, 0.15 for Zone 2A, 0.2 for Zone 2B, 0.3 for Zone 3, and 0.4 for Zone 4;

I = importance factor based on Occupancy Category equal to 1.25 for essential facilities (Occupancy Category I) and hazardous facilities (Occupancy Category II), 1.0 for special occupancy structures (Occupancy Category III) and standard occupancy structures (Occupancy Category IV), and 1.5 for components required for life safety or containing hazardous materials;

C_p = horizontal force factor that ranges from 0.75 to 2.0, dependent on the type of component being designed (the introduced concept of rigid component with rigid anchorage base period less than 0.06 seconds sets C_p = 0.75 and flexible component with rigid anchorage period over 0.06 seconds sets C_p = 2.0. The code has similar language as the 1985 UBC about C_p for flexible or flexibly-anchored equipment but it clearly states that C_p values listed in the table are for rigid nonstructural components, and if C_p is not established by analyses, the values listed in the table should be doubled for flexible or flexibly-anchored nonstructural components but need not exceed 2.0; and

W_p = weight of component.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco located in Seismic Zone 4 (Coastal California) is:

$$F_{p\text{-}ASD} = ZIC_pW_p = 0.4 \times 1.0 \times 2.0W_p = 0.80W_p \tag{3-22}$$

Conversion of the design force to LRFD force level from ASD force level would yield:

$$F_p = 1.4F_{p\text{-}ASD} = 1.4 \times 0.80W_p = 1.12W_p \tag{3-23}$$

The design force for an air-handling unit with rigid anchorage on the roof of the same building in downtown San Francisco located in Seismic Zone 4 is:

$$F_{p\text{-}ASD} = ZIC_pW_p = 0.4 \times 1.0(2.0 \times 0.75)W_p = 0.60W_p \tag{3-24}$$

Note that it is assumed here that the air-handling unit considered for this example is flexible (with a period of longer than 0.06 seconds) and therefore without an established value for C_p, the value of 0.75 per Table 23-P of the code had to be doubled.

Conversion of the design force to LRFD level from ASD level would yield:

$$F_p = 1.4F_{p\text{-}ASD} = 1.4 \times 0.60W_p = 0.84W_p \tag{3-25}$$

1988 NEHRP Provisions

The 1988 *NEHRP Provisions* (FEMA, 1988) for nonstructural seismic design are identical to the 1985 *NEHRP Provisions*.

1991 UBC

The nonstructural seismic design provisions in the 1991 UBC (ICBO, 1991) were reorganized. The force equation for nonstructural seismic design in the 1991 UBC (ICBO, 1991) are the same as in the 1988 UBC.

1991 NEHRP Provisions

The 1991 *NEHRP Provisions* (FEMA, 1992) are similar to the 1985 and 1988 *NEHRP Provisions*. There are two equations in the 1991 *NEHRP Provisions* for seismic design of nonstructural components, one for architectural components and a similar one for mechanical and electrical components. Force levels are at the LRFD or strength level. The component table was reformatted and expanded from the 1988 *NEHRP Provisions*. Cantilevered parapets were added. The mechanical and electrical component equation was changed as well. For architectural components, the equation is:

$$F_p = A_vC_cPW_c \tag{3-26}$$

where:

F_p = lateral seismic force at the LRFD force design level;

A_v = seismic factor representing effective peak velocity-related acceleration and ranges from 0.05 to 0.40;

C_c = seismic factor for architectural components and ranges from 0.6 to 3.0;

P = performance criteria factor and ranges from 0.5 to 1.5 dependent on the Seismic Hazard Exposure Group (Group III is for essential facilities; Group II is for assembly building, larger schools, jails and select other occupancies; and Group I is for all other buildings, including office buildings); and

W_c = weight of component.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco located located on site with $A_v = 0.4$ is:

$$F_p = A_v C_c P W_c = 0.4 \times 3.0 \times 1.5 W_c = 1.80 W_c \tag{3-27}$$

For mechanical and electrical components, the equation is:

$$F_p = A_v C_c P a_c W_c \tag{3-28}$$

where:

F_p = lateral seismic force at the LRFD force design level;

A_v = seismic factor representing effective peak velocity-related acceleration and ranges from 0.05 to 0.40;

C_c = seismic factor for mechanical and electrical component and ranges from 0.67 to 2.00;

a_c = attachment amplification factor related to the component supporting mechanism and taken as 1.0 for fixed (direct) connection, 1.0 for a seismic activated restraining device connection, 1.0 for a resilient support system where $(T_c / T) < 0.6$ or $(T_c / T) > 1.4$, or 2.0 for $0.6 \leq (T_c / T) \leq 1.4$;

T = fundamental period of the building;

T_c = fundamental period of the component; and

W_c = weight of component.

The design force for an air-handling unit with rigid anchorage ($a_c = 1.0$) on the roof of the same building in downtown San Francisco is:

$$F_p = A_v C_c P a_c W_c = 0.4 \times 2.0 \times 0.5 \times 1.0 W_c = 0.40 W_c \tag{3-29}$$

Note that for such a case, a Performance Characteristic Level of "Low" was allowed by the 1991 *NEHRP Provisions*, which would assign a value of 0.5 for P.

1994 UBC

The 1994 UBC (ICBO, 1994) was reorganized into three volumes, with the structural requirements in Volume 2. The 1994 UBC clarified that "equipment" includes major conduit, ducting, and piping. It also required independent support for light fixtures and mechanical services installed in metal suspension systems for acoustical tile and lay-in panel ceilings. The equation for nonstructural seismic design in the 1994 UBC (ICBO, 1994) is the same as in the 1991 UBC, except that the Importance factor, I, for components was renamed I_p and was increased to 1.5 for essential facilities and hazardous facilities.

1994 NEHRP Provisions

The 1994 *NEHRP Provisions* (FEMA, 1995) for nonstructural seismic design were substantially revised from the 1991 *NEHRP Provisions*. The design force for architectural, mechanical and electrical components is obtained by using a single equation (see Eq. 3-30). The previous C_c factor was split into two factors, a_p, which accounts for dynamic interaction between the nonstructural component and the structure, and R_p, which accounts for the ability of the component to absorb energy. Two additional equations provide an upper bound and lower bound for the design force. The force equations are:

$$F_p = (a_p A_p I_p / R_p)\, W_p \quad \text{(primary equation)} \tag{3-30}$$

$$F_p = 4.0 C_a I_p W_p \quad \text{(upper bound equation)} \tag{3-31}$$

$$F_p = 0.5 C_a I_p W_p \quad \text{(lower bound equation)} \tag{3-32}$$

where:

F_p = lateral seismic force at the LRFD force design level;

a_p = component amplification factor that ranges from 1.0 to 2.5 that accounts for amplification due to component flexibility;

A_p = component acceleration factor, which is a function of the fundamental period of the structure and the elevation of the component in the building, ranges from $1.0C_a$ to $4.0C_a$ and is equal to $C_a + (A_r - C_a)(^x/_h)$;

A_r = component acceleration factor at the roof is equal to $2.0A_s \leq 4.0C_a$;

A_s = structure response acceleration factor is equal to $(1.2C_v / T^{2/3}) \leq 2.5C_a$;

x = component elevation in structure relative to grade;

h = average roof elevation of structure relative to grade;

C_v = seismic factor, which is a function of the effective peak velocity-related acceleration, A_v, and the site specific Soil Profile Type (e.g., for A_v = 0.4,

C_v ranges from 0.32 for Soil Profile Type A to 0.96 for Soil Profile Type E);

C_a = seismic factor, which is a function of the effective peak acceleration, A_a, and the site specific Soil Profile Type (e.g., for $A_a = 0.4$, C_a ranges from 0.32 for Soil Profile Type A to 0.44 for Soil Profile Type D);

I_p = importance factor and ranges from 1.0 to 1.5;

R_p = component response modification factor which represents the ability of the component to absorb energy and ranges from 1.0 to 6.0; and

W_p = weight of component.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco is:

$$F_p = (a_p A_p I_p / R_p) W_p = (2.5 \times 4 \times 0.4 \times 1.0 / 1.5) W_p = 2.67 W_p \tag{3-33}$$

$$F_p \leq 4.0 C_a I_p W_p = 4.0 \times 0.4 \times 1.0 W_p = 1.6 W_p \text{ (controls)} \tag{3-34}$$

$$F_p \geq 0.5 C_a I_p W_p = 0.5 \times 0.4 \times 1.0 W_p = 0.2 W_p \tag{3-35}$$

For this example, A_r is assumed to be the maximum of $4.0C_a$, with C_a assumed to be 0.40 (Soil Profile Type D with $A_v = 0.4$).

The design force for an air-handling unit with rigid anchorage ("non-vibration isolated HVAC equipment per Table 3.3.2) on the roof of the same building in downtown San Francisco is:

$$F_p = (a_p A_p I_p / R_p) W_p = (1.0 \times 4.0 \times 0.40 \times 1.0 / 3.0) W_p = 0.53 W_p \text{ (controls)} \tag{3-36}$$

$$F_p \leq 4.0 C_a I_p W_p = 4.0 \times 0.40 \times 1.0 W_p = 1.6 W_p \tag{3-37}$$

$$F_p \geq 0.5 C_a I_p W_p = 0.5 \times 0.40 \times 1.0\ W_p = 0.2 W_p \tag{3-38}$$

1997 UBC

Significant changes were made to the provisions in the 1997 UBC (ICBO, 1997), incorporating concepts from the 1994 *NEHRP Provisions*. As a consequence, the C_p factor was split into two variables, a_p which accounts for dynamic interaction between the nonstructural component and the structure, and R_p which accounts for the ability of the component to absorb energy. An expression was also added to the lateral force formula to account for the amplification of the accelerations in the upper portion of the structure, varying from 1 at grade to 3 at the roof. The amplification limit at the roof was reduced from 4 in the 1994 *NEHRP Provisions* to 3 in the 1997 UBC, based on additional instrumented building records from the 1994 Northridge earthquake. In addition, a major change was made to the design ground shaking estimates in regions of high seismicity. Near-field factors were introduced for sites in close proximity to active faults, which increased the design values of ground shaking by up to 50%. In addition, site factors based on soil type were included in the calculations of the design ground motion for nonstructural components.

There are three force equations for seismic design of nonstructural components with two of the equations serving as upper and lower bounds to the primary equation. With this edition of the UBC, seismic forces were now directly calculated at the strength design (LRFD) level. The 1997 UBC provides the following equation to determine the seismic design force for nonstructural design of components, and equipment and machinery anchorage:

$$F_p = \left(a_p C_a I_p \frac{\left[1 + 3\frac{h_x}{h_r} \right]}{R_p} \right) W_p \tag{3-39}$$

The design force calculated from this equation need not exceed $4.0C_aI_pW_p$ nor be taken less than $0.7C_aI_pW_p$, where:

F_p = lateral seismic force at strength design (LRFD) force design level;

a_p = component amplification factor that ranges from 1.0 to 2.5 that accounts for amplification due to component flexibility;

C_a = seismic factor that depends on Seismic Zone and Soil Profile Type;

I_p = seismic importance factor for components based on Occupancy Category: 1.5 for essential facilities (Occupancy Category I) and hazardous facilities (Occupancy Category II), and 1.0 for special occupancy structures (Occupancy Category III) and standard occupancy structures (Occupancy Category IV);

h_x = component elevation with in the building relative to grade;

h_r = roof elevation of the building relative to grade;

R_p = component response modification factor which represents the ability of the component to absorb energy and ranges from 1.5 to 4.0; and

W_p = weight of component.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco:

$$F_p = \frac{0.4 a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right)W_p \quad (3\text{-}40)$$

$= 1.46W_p$ (controls)

$$F_p \leq 4.0C_aI_pW_p = 4.0 \times 0.44 \times 1.0W_p = 1.76W_p \quad (3\text{-}41)$$

$$F_p \geq 0.7C_aI_pW_p = 0.7 \times 0.44 \times 1.0W_p = 0.31W_p \quad (3\text{-}42)$$

This assumes the default category of Soil Profile Type S_D, and the site is more than 6.2 miles (10 km) away from the San Andreas and Hayward Faults (N_a = 1.0), such that $C_a = 0.44N_a = 0.44$. It should be noted that if the site is in close proximity to a major fault, the value of N_a is taken as 1.5. Compared to the 1994 UBC, the design lateral force a nonstructural component could increase 65% due to changes in the design ground motions alone.

The design force for an air-handling unit with rigid anchorage on the roof of the same building in downtown San Francisco located in Seismic Zone 4 is equal to the design force calculated above for the parapet since per Table 16-O of the 1997 UBC, a_p and R_p for flexible equipment connected to the structure below their center of gravity is 2.5 and 3.0, respectively. However, the design force for the actual anchorage could be different, if an R_p of 1.5 for shallow expansion anchors (length-to-diameter ratio of less than 8), or an R_p of 1.0 for non-ductile anchors or adhesive materials are used. In such cases, the maximum capacity established above ($1.76W_p$) would govern.

1997 NEHRP Provisions

A major change to the design force equations were made in the 1997 *NEHRP Provisions* (FEMA, 1998), adopting the general format of the 1997 UBC provisions. Equations are as follows:

$$F_p = \frac{0.4 a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right)W_p \quad \text{(primary equation)} \quad (3\text{-}43)$$

$$F_p \leq 1.6S_{DS}I_pW_p \quad \text{(upper bound equation)} \quad (3\text{-}44)$$

$$F_p \geq 0.3S_{DS}I_pW_p \quad \text{(lower bound equation)} \quad (3\text{-}45)$$

where:

F_p = lateral seismic force at the LRFD force design level;

a_p = component amplification factor that ranges from 1.0 to 2.5 and accounts for amplification due to component flexibility;

S_{DS} = site specific short-period spectral acceleration;

z = component height in structure relative to grade;

h = average roof height of structure relative to grade;

R_p = component response modification factor which represents the ability of the component to absorb energy and ranges from 1.0 to 3.5;

I_p = component importance factor equal to 1.0 for typical components; and 1.5, if the component is required to function for life safety purposes following an earthquake, contains hazardous materials, a storage rack in occupancies open to the general public, or is anchored to an Seismic Use Group III structure and is needed for continued operation of the facility; and

W_p = weight of component.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco:

$$F_p = \frac{0.4 a_p S_{DS}}{\left(\dfrac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right)W_p \qquad (3\text{-}46)$$

$$= \frac{0.4\times 2.5\times 1.0}{\left(\dfrac{2.5}{1.0}\right)}\left(1+2\frac{h}{h}\right)W_p = 1.20W_p \quad \text{(controls)}$$

$$F_p \leq 1.6 S_{DS} I_p W_p = 1.6 \times 1.0 \times 1.0 W_p = 1.6 W_p \qquad (3\text{-}47)$$

$$F_p \geq 0.3 S_{DS} I_p W_p = 0.3 \times 1.0 \times 1.0 W_p = 0.3 W_p \qquad (3\text{-}48)$$

This assumes the default category of Soil Profile Type D, and a site location corresponding to 37.7887 °N and 122.4005 °W (55 Second Street in San Francisco) such that the mapped value of S_{DS} for the site is 1.00.

The design force for an air-handling unit with rigid anchorage (“non-vibration isolated HVAC equipment per Table 3.3.2 of the guideline) on the roof of the same building in downtown San Francisco is:

$$F_p = 0.4 \times 1.0 \times 1.0(1 + 2^{h}/_{h}) / (2.5 / 1.0)W_p = 0.48W_p \quad \text{(controls)} \qquad (3\text{-}49)$$

$$F_p \leq 1.6 S_{DS} I_p W_p = 1.6 \times 1.0 \times 1.0 W_p = 1.6 W_p \qquad (3\text{-}50)$$

$$F_p \geq 0.3 S_{DS} I_p W_p = 0.3 \times 1.0 \times 1.0 W_p = 0.3 W_p \qquad (3\text{-}51)$$

2000 International Building Code

In 1997, International Code Council (ICC) consisting of representatives of Building Officials and Code Administrators International (BOCA), the International Conference of Building Officials (ICBO), and the Southern Building Code Congress

International, Inc. (SBCCI) drafted a set of regulations for building systems consistent with and inclusive of the scope of the existing model codes. This effort resulted in the first edition of the *International Building Code* (ICC, 2000) which replaced the UBC and the other model buildings codes. For seismic design of nonstructural components, the 2000 IBC adopted the procedures of the 1997 UBC.

2000 NEHRP Provisions

The equations for nonstructural seismic design in the 2000 *NEHRP Provisions* (FEMA, 2001) were identical to those in the 1997 *NEHRP Provisions*, except for very minor changes in the R_p values. In addition, a fourth equation was added that could be used with modal analysis of the building. The fourth equation is:

$$F_p = ((a_i\ a_p) / (R_p / I_p))A_x W_p \tag{3-52}$$

Where the parameters have the definition as in Eq. 3-43, and in addition:

a_i = acceleration at level i obtained from modal analysis;

A_x = torsional amplification factor.

ASCE/SEI 7-02

Building codes began a transition from an all-inclusive document into which text from different reference standards was transcribed, to a document that adopted standards by reference. To support the transition, ASCE/SEI 7-02, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2002), a revision of the original 1998 edition of the standard, was updated to incorporate the seismic provisions of 2000 *NEHRP Provisions*.

3.1.3 Unified Nonstructural Requirements 2003-2016

In 2003, the *International Building Code* adapted a standard by reference for seismic design. From this point forward, revisions to the design procedures for nonstructural components were made in ASCE/SEI 7.

2003 International Building Code

In the 2003 IBC (ICC, 2003), the provisions covering seismic design of nonstructural components were removed from the building code, and the requirements adopted by reference to ASCE/SEI 7-02. Subsequent editions of the IBC adopt the version of ASCE/SEI 7 in force at the time of publication by reference.

2003 NEHRP Provisions

Equations in the 2003 *NEHRP Provisions* (FEMA, 2004a) are identical to those in the *2000 NEHRP Provisions* except that a new equation based on the component period could be used to calculate a need-not-exceed value for the design force. The commentary to the 2003 *NEHRP Provisions* (FEMA, 2004b) explains the basis of the maximum value. The structural response spectrum begins to reduce beyond a transition period of S_{D1} / S_{DS}, and studies have shown a similar reduction in nonstructural component amplification at periods about 25% larger. The primary, and upper and lower bound equations for seismic design of nonstructural components, and machine and equipment anchorage are:

$$F_p = \frac{0.4 a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right)W_p \quad \text{(primary equation)} \tag{3-53}$$

$$F_p \leq 1.6 S_{DS} I_p W_p \tag{3-54}$$

Exception: If the component period, T_p, is greater than T_{flx}, where T_{flx} = (1 + 0.25 $^z/_h$)S_{D1} / S_{DS}, the value of F_p may be reduced by the ratio of T_{flx} / T_p) in the above two equations.

$$F_p \geq 0.3 S_{DS} I_p W_p \tag{3-55}$$

where:

F_p = lateral seismic force at the LRFD force design level;

a_p = component amplification factor that ranges from 1.0 to 2.5 and accounts for amplification due to component flexibility;

S_{DS} = site specific short-period design spectral acceleration;

S_{D1} = site specific one-second design spectral acceleration;

z = component elevation in structure relative to grade;

h = roof elevation in structure relative to grade;

R_p = component response modification factor which represents the ability of the component to absorb energy and ranges from 1.0 to 12;

I_p = component importance factor typically equal to 1.0, but if the component is required to function for life safety purposes following an earthquake, the component contains hazardous materials, or the component is in an Occupancy Category IV structure and is needed for continued operation of the facility the importance factor is equal to 1.5;

T_p = component fundamental period;

T_{flx} = transition point at which nonstructural force reduction begins to reduce as the structural response spectrum begins to diminish; and

W_p = weight of component.

The fourth equation for using modal analysis to calculate the seismic design force is exactly the same as it appears in the 2000 *NEHRP Provisions*.

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco:

$$F_p = \frac{0.4 a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right)W_p \quad \text{(primary equation)} \tag{3-56}$$

$$= \frac{0.4\times 2.5\times 1.0}{\left(\frac{2.5}{1.0}\right)}\left(1+2\frac{h}{h}\right)W_p = 1.20W_p \quad \text{(controls)}$$

$$F_p \le 1.6 S_{DS} I_p W_p = 1.6 \times 1.0 \times 1.0 W_p = 1.6 W_p \tag{3-57}$$

$$F_p \ge 0.3 S_{DS} I_p W_p = 0.3 \times 1.0 \times 1.0 W_p = 0.3 W_p \tag{3-58}$$

This assumes the default category of Soil Profile Type S_D, and a site location corresponding to 37.7887° N and 122.4005° W (55 Second Street in San Francisco) such that the mapped values for the site are S_{DS} = 1.000 and S_{D1} = 0.632. The building is assumed to be a 20-story steel moment frame with a fundamental period of well over $T_{flx} = (1 + 0.25\ ^{z}/_{h})(S_{D1} / S_{DS}) = (1 + 0.25\ (1))(1.000 / 0.632) = 0.79$ seconds, but the parapet period T_p is well under T_{flx} = 0.79 seconds so that there is no reduction per the exception.

The design force for an air-handling unit with rigid anchorage ("non-vibration isolated HVAC equipment" per Table 3.3.2) on the roof of the same building in downtown San Francisco would be:

$$F_p = 0.4 \times 1.0 \times 1.0(1 + 2\ ^{h}/_{h}) / (2.5 / 1.0)W_p = 0.48W_p \text{ (controls)} \tag{3-59}$$

$$F_p \le 1.6 S_{DS} I_p W_p = 1.6 \times 1.0 \times 1.0 W_p = 1.6 W_p \tag{3-60}$$

$$F_p \ge 0.3 S_{DS} I_p W_p = 0.3 \times 1.0 \times 1.0 W_p = 0.3 W_p \tag{3-61}$$

Similar to the parapet example, since the period T_p of a rigidly-anchored air-handling unit is typically under T_{flx} = 0.79 seconds, there is no reduction per the exception.

ASCE/SEI 7-05

The nonstructural provisions of ASCE/SEI 7-05 including Supplement No. 1, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2006), maintained the same design approach and force equations found in the prior edition, SEI/ASCE 7-02. It should be noted that Supplement No. 1 was included in the initial release as ASCE/SEI 7-05. While very similar to the requirements in the 2003 *NEHRP Provisions*, the need-not-exceed equation for F_p based on the component period was not adopted. Changes were made to the organization and seismic factors in Table 13.6-1 for mechanical components, based in part on the concepts of flexible and rigid equipment, and ductile and rugged behavior drawn from the commentary of the *Recommended Lateral Force Requirements and Commentary*, (SEAOC, 1999) RP-812, *Practical Guide to Seismic Restraint* (ASHRAE, 1999). ASCE/SEI 7-05 was adopted by reference into the 2006 IBC (ICC, 2006) and the 2009 IBC (ICC, 2009).

2009 NEHRP Provisions

In the 2009 *NEHRP Provisions* (FEMA, 2009), ASCE/SEI 7-05 was used as a reference document and changes were proposed. No changes were proposed for the equations used in the seismic design of nonstructural components.

The Commentary to Chapter 13 of the 2009 *NEHRP Provisions*, Seismic Design Requirements for Nonstructural Components, contained an expanded discussion regarding performance objectives for nonstructural components, which included the following:

> "While specific performance goals for nonstructural components have yet to be defined in the building codes, the component importance factor (I_p) implies performance levels for certain cases. For noncritical nonstructural components (those with an importance factor, I_p, or 1.0) the following behaviors are anticipated for shaking having different levels of intensity:

1. minor earthquake ground motion—minimal damage not likely to affect functionality;
2. moderate earthquake ground motions—some damage that may affect functionality; and
3. design earthquake ground motions—major damage but significant falling hazards are avoided; likely loss of functionality.

Components with importance factor greater than 1.0 are expected to remain in place, sustain limited damage, and when necessary, function following an earthquake."

These performance objectives represent the de facto performance objectives underlying code provisions in ACSE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010).

Requirements for the design of nonstructural components in new and existing structures share many common attributes, although they are covered in different standards. The most current requirements for new buildings are found in ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other*

*Structures, while t*he requirements for existing buildings are found in ASCE/SEI 41-13, *Seismic Evaluation and Retrofit of Existing Buildings* (ASCE, 2014b).

2015 NEHRP Provisions

The 2015 *NEHRP Provisions* (FEMA, 2015b) for nonstructural seismic design are identical to the 2009 *NEHRP Provisio*ns.

3.2 Seismic Design Provisions for Nonstructural Components in ASCE/SEI 7-10 and ASCE/SEI 7-16

The seismic design provisions for new structures are currently contained in Chapter 13 of ASCE/SEI 7-10 including Supplement No. 1, since significant changes for the anchorage of nonstructural components are included in Supplement No. 1. The next edition of the *International Building Code* in 2018 will reference ASCE/SEI 7-16, which represents the latest iteration of the nonstructural procedures that been continuously developed since their first appearance in the 1997 UBC. While this section focuses on the provisions of ASCE/SEI 7-16, the discussion is generally applicable to ASCE/SEI 7-10 including Supplement No. 1 as well. Significant differences between the two editions are presented.

The nonstructural provisions of ASCE/SEI 7-16 are organized in six sections, covering information on the applicability of the nonstructural design provisions, determination of the relative importance of the component and methods for establishing compliance with the standard, procedures for determining acceleration and displacement demands, design of anchorage of components to the structure, and detailed requirements for architectural, mechanical, and electrical components.

Components are classified by importance; those identified as critical to life safety are classified as "designated seismic systems" and are subject to more stringent design and quality assurance requirements. Smaller non-critical components may be completely exempt from the nonstructural seismic design based on the weight, size (for distribution systems), and mounting configuration. Most furniture and building contents are exempt from the seismic provisions of ASCE/SEI 7-16.

Unlike primary structural systems, where earthquake performance targets are based on the probability of collapse when subjected to the risk-targeted maximum considered earthquake (MCE_R) ground motions, nonstructural component designs are based on behavior at the design earthquake (DE) ground motion level, a strong but more frequent earthquake taken as two-thirds of the MCE_R ground motions. Component-specific nonstructural performance goals have not been defined, but general performance expectations are provided, similar to those outlined in the 2009 *NEHRP Provisions*. For non-critical components, significant damage is expected when the structure is subject to DE ground motions, but serious falling hazards are avoided. Loss of functionality is expected. There are no implicit performance goals associated with the MCE_R for nonstructural components. For designated seismic systems, components are expected to remain in place and function following DE ground motions. Qualitative goals for performance in lower ground shaking intensities are provided, but neither the ground shaking intensities nor component performance expectations are explicitly defined. Critical components classified as designated seismic systems are expected to remain in place, sustain limited damage and function following an earthquake, if necessary.

Several options are available for establishing compliance with the nonstructural requirements, including project-specific design and documentation, and certification by the manufacturer. Designated seismic systems may be subject to special certification requirements if they must remain operable following the design earthquake or if they contain more than a specified amounts of a hazardous substance. Special certification may require shake table testing or more advanced analysis to establish compliance.

Acceleration demands on nonstructural components may be determined using several approaches. The most commonly used formula (ASCE/SEI 7-16 Eq. 13.3-1) accounts for variation of acceleration with relative height within the structure, without regard for the specific nature or dynamic properties of the structure:

$$F_p = \frac{0.4 a_p S_{DS}}{\left(\dfrac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right)W_p \qquad (3\text{-}62)$$

where:

F_p = horizontal equivalent static seismic design force acting at the center of gravity of component and distributed relative to mass distribution of component;

a_p = component amplification factor (between 1.0 and 2.5) as tabulated in ASCE/SEI 7-16 Table 13.5-1 for architectural components and ASCE/SEI 7-16 Table 13.6-1 for mechanical and electrical components;

S_{DS} = five-percent-damped spectral response acceleration parameter at short-period as defined in ASCE/SEI 7-16 Section 11.4.4;

R_p = component response modification factor and ranges between 1.0 to 12.0 as tabulated in ASCE/SEI 7-16 Table 15.5-1 for architectural components and ASCE/SEI 7-16 Table 13.6-1 for mechanical and electrical components;

I_p = component importance factor and is either 1.0 or 1.5 as indicated in ASCE/SEI 7-16 Section 13.1.3;

z = height with building of the point of anchorage of the component with respect to the base;

h = average roof height of building but not more than height of seismic force-resisting system of structure with respect to base; and

W_p = weight of component.

In developing the acceleration and displacement demand provisions, consideration was given to the reality that it in many cases, design information on the building seismic force-resisting system may not be available for use in the design of the nonstructural components. As discussed in Section 3.3, it is relatively rare that the structural engineer in charge of the building design is also involved in the design of the nonstructural components. Therefore, the individuals who are responsible for the nonstructural design may not have access to detailed information about the dynamic properties such as period or modes shapes, or even the type of lateral-force resisting system of the building. For nonstructural designs associated with renovations of existing buildings, detailed structural information will rarely be known. In addition, dynamic properties of nonstructural components and equipment such as the fundamental period are not provided by the manufacturer. In the case of built-up components, the design engineer would have difficulty determining the fundamental period. Therefore the equations for acceleration and displacement demands were developed in light of the information likely to be available to the design engineer.

Since the introduction of the first form of ASCE/SEI 7-16 Eq. 13.3-1 in the 1994 *NEHRP Provisions*, there has been debate as to the merits of this equation, especially with regard to the term intended to capture the amplification of accelerations over the height of the building. This term, $(1 + 2\,{}^{z}/_{h})$, was determined based on the examination of instrumented buildings subject to peak ground accelerations of 0.10g or higher, available circa 1995. The term approximates the mean plus one standard deviation of the recorded floor accelerations for a wide range of structures. Critics of the formula have used computer models and analysis of individual instrumented structures to suggest that Eq. 13.3-1 produces inaccurate and usually unrealistically high demands. The majority of these studies have focused on taller, longer period structures which, while important, are not representative of the majority of structures that make up the built environment. The studies have in general focused on the force equation, ASCE/SEI 7-16 Eq. 13.3-1, and have not considered alternative equations included in ASCE/SEI 7-16.

In addition to force equation ASCE/SEI 7-16 Eq. 13.3-1, there are dynamic analysis methods available for determining building-specific values of the seismic design force, F_p, if the dynamic properties of the building are known. The equation is intended for use on base-isolated and long period buildings where the seismic design forces are expected to be lower than those given by the generic F_p equation (ASCE/SEI 7-16 Eq. 13.3-1). A nonstructural force equation that utilizes the linear dynamic analysis procedures of Section 12.9 in ASCE/SEI 7-16 is provided:

$$F_p = \frac{a_i a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)} A_x \tag{3-63}$$

where a_i is the acceleration at level i obtained from the modal analysis and A_x is the torsional amplification factor determined by ASCE/SEI 7-16 Eq.12.8-14. The value of the Response Modification factor of the structure, R_p, is taken as 1.0 when these procedures are used to determined nonstructural forces. An examination of whether ASCE/SEI 7-16 Eq. 13.3-4 provides a more satisfactory approximation of floor accelerations in taller or longer period structures is generally not included in studies examining the accuracy and suitability of the ASCE/SEI 7-16 nonstructural force computations.

In ASCE/SEI 7-16, two additional options for determination of component accelerations were added. The first utilizes nonlinear response history procedures of Chapters 16, 17, and 18 in ASCE/SEI 7-16, and the second uses floor response spectra methods. Where seismic response history analysis is used with at least seven ground motions, the design floor acceleration at Level i, a_i, is taken as the average peak floor acceleration computed from the seven ground motions. When less than seven ground motions are used, the design floor acceleration for each floor is the peak value obtained over all the ground motions analyzed.

Application of the floor response spectra approach requires that a floor response spectrum be calculated at each level of the structure, for each design earthquake ground motion record analyzed. The value of the component amplification factor, a_p, is taken as 1.0. The floor acceleration, a_i, for a given component is the maximum acceleration value from the floor response spectra for the component period.

An alternate floor response spectra method based on a modal analysis is also available in ASCE/SEI 7-16 Section 13.3.1.2. The periods and mode shapes of the structure are calculated for at least the first three modes in each orthogonal direction using the modal analysis procedure in ASCE/SEI 7-16 Section 12.9. The component dynamic amplification factor, D_{AF}, is a function of the ratio of the component period, T_p, to the building modal period, T_x, and is found using ASCE/SEI 7-16 Figure 13-1 (shown here as Figure 3-1):

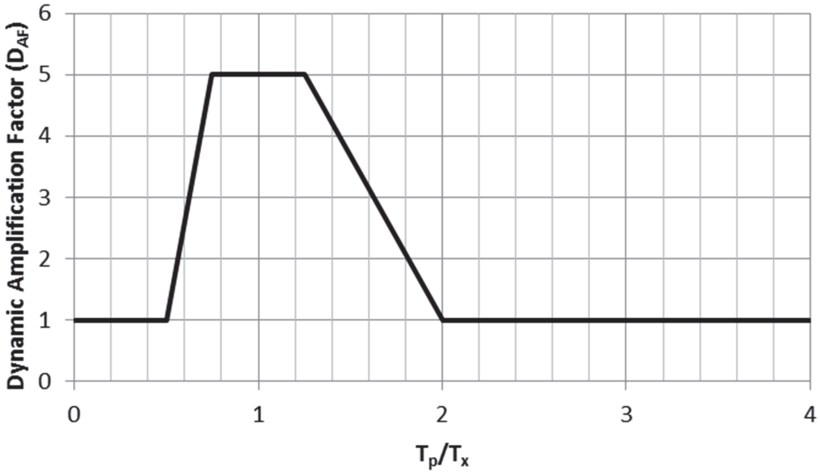


Figure 3-1 Component dynamic amplification factor.

For each of the first three modes in each direction, the modal acceleration for components at each floor is calculated using ASCE/SEI 7-16 Eq. 13.3-5:

$$A_{ix} = p_{ix}S_{ai}D_{AF} \tag{3-64}$$

where:

A_{ix} = floor acceleration for mode x at Level i;

p_{ix} = modal participation factor for mode x at Level i obtained from the modal analysis;

S_{ai} = spectral acceleration for mode x; and

D_{AF} = dynamic amplification factor as a function of the ratio of component period to building period for mode x using Figure 3-1.

The floor response spectrum is taken as the maximum floor acceleration at each building modal period for at least the first three modes, but not less than the spectral acceleration at the base of the building. The design seismic horizontal force in each direction for a nonstructural component is determined by ASCE/SEI 7-16 Eq. 13.3-4 with the product of $a_i a_p$ replaced by A_{ix}, the acceleration from the floor response spectrum for the period of vibration of the nonstructural component at the Level i on which the nonstructural component is anchored.

All these above approaches are subject to maximum and minimum design forces.

$$F_{p\text{-}max} = 1.6S_{DS}I_pW_p \tag{3-65}$$

$$F_{p\text{-}min} = 0.3S_{DS}I_pW_p \tag{3-66}$$

For the purposes of computing the acceleration demands, all components are characterized by two variables, a component response modification factor R_p, used to represent the energy absorption capability of the component and anchorage, and a component amplification factor a_p, which is intended to capture the dynamic amplification of the component response as function of the fundamental period of the structure and the period of the component. Tabulated values of a_p and R_p included in the standard cover common nonstructural components, and are based primarily on engineering judgment.

A number of recent studies suggest that the value of a_p, which is capped at 2.5, should in fact be significantly larger for some combinations of building and component dynamic properties. Similar studies were considered during the development of the current design provisions, but the cap on a_p was considered appropriate for several reasons. First, most of the dynamic interaction studies were based on the assumption of linear behavior of both the structure and the component. Large amplifications resulted when the period of the component approached the period of one of the lower modes of the structure. The committee believed that during strong ground shaking, both the structure periods and the period of the nonstructural components will shift as the building undergoes damage, which could either reduce or increase amplification in a manner that cannot be predicted. Second, it was felt that if a component is exposed to very high dynamic amplification, the effects would likely be transient as the component period will shift if yielding occurs. Finally, if the high levels of amplification predicted due to resonance were commonly occurring, one would expect a pattern of significant damage to some code-conforming anchored nonstructural components, even in moderately strong shaking, and this has not generally been observed.

Formulas for determining displacement demands on nonstructural components due to story drift and relative displacements for components spanning between adjacent structures are provided. Due to the limited availability of structure-specific story drift data to the designer of the nonstructural components, the procedures default to the story drift limits of ASCE/SEI 7-16. Structure-specific drift data may be used if available. Guidance for consideration of component displacements between lateral supports and potential interaction between adjacent components or structural elements is minimal.

Positive anchorage of the component to the structure is required, and friction due solely to the effects of gravity cannot be used to resist seismic forces. An exception is made for roof-mounted solar photovoltaic arrays, which are permitted to rely on friction due to gravity provided they meet specific requirements.

One of the basic tenets of the current design procedures has been to avoid brittle failures of anchorage to the structure, especially anchors in concrete and masonry. The lateral systems for many nonstructural components are statically determinate, which means the failure of a single anchorage can result in loss of stability. To preclude anchorage failures, earlier versions of the nonstructural design procedures amplified connection forces by a factor of 1.3, and there were additional adjustments to increase the design loads on anchors in concrete and masonry to ensure that either the anchor was strong enough to resist DE loading without failure, or that a yielding element was present in the load path preventing an anchor failure.

Concurrent with developments of the nonstructural anchorage design provisions, there was a complete revision of the design methodologies for cast-in-place and post-installed anchors in concrete and masonry. The design and qualification procedures for anchors are now far more complex, and the assumed design capacities of anchors have been reduced substantially. Harmonizing the nonstructural anchorage provisions of the different editions of ASCE/SEI 7 with the evolving anchor design provisions in the concrete and masonry standards is an ongoing process.

Component-specific design provisions are included for some, but not all of the architectural, mechanical and electrical components for which tabulated design factors are provided. ASCE/SEI 7-16 references industry standards for many components, in some cases amending them to achieve the desired seismic performance level. Specific requirements are included for select architectural components including exterior nonstructural wall elements, suspended ceilings, access floors, exit stairs, and glazing. For mechanical and electrical components, specific requirements are included for electrical components, rooftop solar photovoltaic arrays, and distribution systems (piping, ducts, and raceways).

An example calculation for the design force of a cantilevered rooftop parapet of an office building in downtown San Francisco is:

$$F_p = \frac{0.4 a_p S_{DS}}{\left(\dfrac{R_p}{I_p}\right)}\left(1+2\frac{z}{h}\right)W_p \qquad (3\text{-}67)$$

$$= \frac{0.4\times 2.5\times 1.0}{\left(\dfrac{2.5}{1.0}\right)}\left(1+2\frac{h}{h}\right)W_p = 1.20W_p \quad \text{(controls)}$$

$$F_p \le 1.6 S_{DS} I_p W_p = 1.6 \times 1.0 \times 1.0 W_p = 1.6 W_p \qquad (3\text{-}68)$$

$$F_p \ge 0.3 S_{DS} I_p W_p = 0.3 \times 1.0 \times 1.0 W_p = 0.3 W_p \qquad (3\text{-}69)$$

This assumes the default category of Soil Profile Type S_D, and a site location corresponding to 37.7887 °N and 122.4005 °W (55 Second Street in San Francisco) such that the mapped values for the site are S_{DS} = 1.000 and S_{D1} = 0.632. ASCE/SEI 7-05 uses the 2002 USGS seismic mapping updates.

The design force for an air-handling unit with rigid anchorage on the roof of the same building in downtown San Francisco is:

$$F_p = 0.4 \times 2.5 \times 1.0W_p(1 + 2^h/_h) / (^{6.0}/_{1.0}) = 0.50W_p \quad \text{(controls)} \tag{3-70}$$

$$F_p \leq 1.6S_{DS}I_pW_p = 1.6 \times 1.0 \times 1.0W_p = 1.6W_p \tag{3-71}$$

$$F_p \geq 0.3S_{DS}I_pW_p = 0.3 \times 1.0 \times 1.0W_p = 0.3W_p \tag{3-72}$$

3.3 Comparison of Nonstructural Component Design Force Levels

Summaries of design force for cantilevered rooftop parapet and rigidly-anchored air-handling unit examples are shown in Table 3-1 and Table 3-2, respectively.

Table 3-1 Comparison of Nonstructural Component Design Force Levels in Various Code Resource Documents for a Cantilevered Rooftop Parapet in San Francisco

Edition[a]	UBC/IBC	NEHRP	ASCE/SEI 7
1927	$0.14W_p$	--	--
1935-76	$1.40W_p$	--	--
1979-82	$1.12W_p$	--	--
1985	$1.12W_p$	$0.54W_p$	--
1988	$1.12W_p$	$0.54W_p$	$1.12W_p$
1991	$1.12W_p$	$1.80W_p$	--
1993	--	--	$1.80W_p$
1994	$1.12W_p$	$1.60W_p$	--
1995	--	--	$1.60W_p$
1997	$1.46W_p$	$1.20W_p$	--
2000	[b]	$1.20W_p$	--
2002	--	--	$1.20W_p$
2003	[b]	$1.20W_p$	--
2005(1)	--	--	$1.20W_p$
2006-09	[b]	$1.20W_p$	--
2010	--	--	$1.20W_p$
2010(1)	--	--	$1.20W_p$
2012	[b]	--	--
2015	[b]	$1.20W_p$	--
2016	--	--	$1.20W_p$

[a] Numbers in brackets refer to inclusion of supplement(s) for that edition.
[b] 2000 IBC used same provisions as the 1997 UBC, while later versions of the IBC referenced the in-force edition of ASCE/SEI 7.

Table 3-2 Comparison of Nonstructural Component Design Force Levels in Various Code Resource Documents for a Rooftop Air-Handling Unit in San Francisco

Edition[a]	UBC/IBC	NEHRP	ASCE/SEI 7
1927	--	--	--
1935-73	--	--	--
1976	$0.63W_p$	--	--
1979-82	$0.42W_p$	--	--
1985	$0.42W_p$	$0.80W_p$	--
1988	$0.84W_p$	$0.80W_p$	$0.84W_p$
1991	$0.84W_p$	$0.40W_p$	--
1993	--	--	$0.4W_p$
1994	$0.84W_p$	$0.53W_p$	--
1995	--	--	$0.53W_p$
1997	$1.46W_p$	$0.48W_p$	--
2000	[b]	$0.48W_p$	--
2002	--	--	$0.48W_p$
2003	[b]	$0.48W_p$	--
2005(1)	--	--	$0.50W_p$
2006-09	[b]	$0.48W_p$	--
2010	--	--	$0.50W_p$
2010(1)	--	--	$0.50W_p$
2012	[b]	--	--
2015	[b]	$0.48W_p$	--
2016	--	--	$0.50W_p$

[a] Numbers in brackets refer to inclusion of supplement(s) for that edition.
[b] 2000 IBC used same provisions as the 1997 UBC, while later versions of the IBC referenced the in-force edition of ASCE/SEI 7.

3.4 Nonstructural Component Anchorage

Design of supports and anchorage is required for all nonstructural components in Seismic Design Category D or higher. A great many nonstructural components are anchored to concrete slabs and walls, or masonry walls. Post-installed anchors are often preferred for the anchorage of nonstructural components, due to both the difficulty of accurately locating cast-in-place anchors, and because in many cases the exact nonstructural component has not been selected when the slabs or walls are constructed. Prior to the 1994, post-installed anchors were designed to the calculated seismic force on the component without modification, and allowable anchor capacities were based on static tests with a factor of safety of four for anchors installed with special inspection and eight if installed without special inspection. The determination of post-installed anchor capacities is discussed in greater detail in Section 3.5.

Following reports of nonstructural of component damage due to failures of anchors in the 1994 Northridge earthquake, both the anchor capacities and the calculated seismic demands were reexamined. Over the next two decades, there were substantial changes in the design of nonstructural component anchorage, with reduction in anchor capacities under seismic loading, and increased anchor design forces. The provisions for the anchorage of nonstructural components are found in Section 13.4.2 of ASCE/SEI 7-16. The design provisions for anchors to concrete and masonry are found in this section, as well as in the *International Building Code*.

In ASCE/SEI 7-05, the design forces for all anchors are amplified by a factor of 1.3 unless the anchors are designed for the maximum force that could be transferred to the anchor from the component (e.g., as limited by rupture of sheet metal). In addition, the value of R_p used for the design of anchors to concrete and masonry is limited to 1.5, unless the design of the anchorage is governed by the strength of a ductile steel element, or post-installed anchors pre-qualified for seismic applications in accordance with ACI 355.2-07, *Qualification of Post-Installed Mechanical Anchors in Concrete and Commentary* (ACI, 2007), are used and the anchors are designed to be ductile in accordance with Appendix D in ACI 318-05, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2005). As originally published, ASCE/SEI 7-05 Section 13.4.2, incorrectly requires all three of these conditions: (1) the component anchorage is designed to be governed by the strength of a ductile steel element; (2) the design of post-installed anchors in concrete used for the component anchorage is prequalified for seismic applications in accordance with ACI 355.2; and (3) the anchor is designed in accordance with Section 14.2.2.14 to be met to allow for an R_p greater than 1.5. In the errata published for ASCE 7-05, the wording was changed so that only one of the three requirements had to be met to allow for an R_p greater than 1.5. The problem with this correction in the errata is that the requirement for qualification in accordance with ACI 355.2 is no longer tied to the requirement for ductile design in accordance with ACI 318-05. Thus, for post-installed anchors, although they could not be designed for ductile failure in most cases, a value greater than 1.5 for R_p could be used as long as they are qualified. Qualification testing in ACI 355.2-07 (and under AC193, *Acceptance Criteria for Mechanical Anchors in Concrete Elements* (ICC-ES, 2015)) did not require ductile failure of the anchor.

The 2006 IBC (ICC, 2006) adopted by reference ASCE/SEI 7-05 including Supplement No. 1 (ASCE, 2006), while the 2009 IBC adopted by reference ASCE/SEI 7-05 including Supplement No. 1 and Supplement No. 2 (ASCE, 2008). Supplement No. 2 to ASCE/SEI 7-05 affected only the minimum base shear for structures and made no changes to the nonstructural provisions. Regardless of whether the 2006 IBC or the 2009 IBC was used, the provisions for nonstructural component anchorage in concrete or masonry are problematic.

The 2012 IBC (ICC, 2012) adopted by reference ASCE/SEI 7-10. There were significant changes to the requirements for anchorage provided in the 2009 IBC. The force amplification factor of 1.3 and the R_p limit of 1.5 for anchorage were replaced by a limit of 6 for the value of R_p when calculating forces for anchorage. Anchors in concrete are designed in accordance with Appendix D in ACI 318-11, *Building Code Requirements for Structural Concrete and Commentary* (ACI 2011b). Anchors in masonry are to be designed in accordance with ACI 530/530.1-11, *Building Code Requirements and Specification for Masonry Structures and Related Commentaries* (ACI, 2011a), with an exception stating that unless the anchorage between the component and the building underwent ductile yielding at load levels corresponding to anchor forces less than the anchor design strength, the design strength of the anchors has to be 2.5 times greater than the factored loads to the anchor. The requirements of Appendix D in ACI 318-11, as modified in Section 1905.1.9 of the 2012 IBC, require design of anchors to be governed by the steel strength of a ductile steel element.

The 2015 IBC adopted by reference ASCE/SEI 7-10 including Supplement No.1. The requirements for anchorage to concrete were again revised in Supplement No. 1, with the introduction of an overstrength factor for anchors, to be applied if required by ACI 318-11 Appendix D. The value of Ω_0 ranges from 1.5 to 2.5.

In ASCE/SEI 7-16, anchors installed in masonry are permitted to use the tabulated values of Ω_0 rather than a constant 2.5 multiplier for anchors when the anchor force is not governed by a ductile yielding element. In addition, the maximum value for Ω_0 has been reduced from 2.5 to 2.0.

The horizontal design force, F_p, for nonstructural components expressed as a fraction of gravity, g, using the provisions of ASCE/SEI 7-05, ASCE/SEI 7-10, ASCE/SEI 7-10 including Supplement No. 1, and ASCE/SEI 7-16 are summarized in Table 3-3. The values for F_p are given for the component itself, which have not changed from edition to edition and the values of F_p for design of anchorage to concrete where behavior is governed by ductile yielding of the anchorage, and cases where it is not. It is assumed that S_{DS} = 1.0 and I_p = 1.0, and that the component is located on the roof of the structure, $^z/_h$ = 1.0.

In general, forces for the design of ductile anchors have remained fairly constant (see column of data in Table 3-3 providing the relative ductile anchor force for comparison to the value established by ASCE/SEI 7-05), while there have been significant variations in the design demands for nonductile anchors (see column of data in Table 3-3 providing the relative nonductile anchor force for comparison to the value established by ASCE/SEI 7-05). Forces for nonductile anchors rose significantly in ASCE/SEI 7-10, and have been reduced by 20% in ASCE/SEI 7-16 for most components.

Table 3-3 Component and Anchorage Design Forces

	a_p	R_p	Design Force F_p	Ductile Anchor F_p	Ω_0	Nonductile Anchor F_p	Relative Ductile Anchor Force[1]	Relative Nonductile Anchor Force[2]
ASCE/SEI 7-05 and Supplement No. 1	2.5	9	0.33	0.43	N.A.	2.08	1.00	1.00
	2.5	6	0.50	0.65	N.A.	2.08	1.00	1.00
	2.5	3	1.00	1.30	N.A.	2.08	1.00	1.00
	2.5	2.5	1.20	1.56	N.A.	2.08	1.00	1.00
	2.5	2	1.50	1.95	N.A.	2.08	1.00	1.00
	1	2.5	0.48	0.62	N.A.	1.04	1.00	1.00
	1	1.5	0.80	1.04	N.A.	1.04	1.00	1.00
ASCE/SEI 7-10	2.5	9	0.33	0.50	N.A.	1.25	1.16	0.60
	2.5	6	0.50	0.50	N.A.	1.25	0.77	0.60
	2.5	3	1.00	1.00	N.A.	2.50	0.77	1.20
	2.5	2.5	1.20	1.20	N.A.	3.00	0.77	1.44
	2.5	2	1.50	1.50	N.A.	3.75	0.96	1.80
	1	2.5	0.48	0.48	N.A.	1.20	0.77	1.15
	1	1.5	0.80	0.80	N.A.	2.00	0.77	1.92
ASCE/SEI 7-10 and Supplement No. 1	2.5	9	0.33	0.50	2.5	1.25	1.16	0.60
	2.5	6	0.50	0.50	2.5	1.25	0.77	0.60
	2.5	3	1.00	1.00	2.5	2.50	0.77	1.20
	2.5	2.5	1.20	1.20	2.5	3.00	0.77	1.44
	2.5	2	1.50	1.50	2.5	3.75	0.96	1.80
	1	2.5	0.48	0.48	2.5	1.20	0.77	1.15
	1	1.5	0.80	0.80	1.5	1.20	0.77	1.15
ASCE/SEI 7-16	2.5	9	0.33	0.50	2	1.00	1.16	0.48
	2.5	6	0.50	0.50	2	1.00	0.77	0.48
	2.5	3	1.00	1.00	2	2.00	0.77	0.96
	2.5	2.5	1.20	1.20	2	2.40	0.77	1.15
	2.5	2	1.50	1.50	2	3.00	0.96	1.44
	1	2.5	0.48	0.48	2	0.96	0.77	0.92
	1	1.5	0.80	0.80	1.5	1.20	0.77	1.15

[1] F_p for a ductile anchor divided by the ASCE/SEI 7-05 F_p using for the same a_p and R_p.
[2] F_p for a nonductile anchor divided by the ASCE/SEI 7-05 F_p using for the same a_p and R_p.

3.5 Brief History of U.S. Code Provisions for Anchorage to Concrete

The origins of allowable loads for anchor bolts that were in use prior to 2000 (e.g., as provided in Table 26-G of the 1976 UBC) are not generally known, nor are the authors aware of the basis for the safety factors traditionally applied to the average ultimate anchor capacities as measured in monotonic tension and shear tests in mass concrete. Although testing of anchors in concrete did not support the stated safety factors in the 1976 UBC, the allowable loads provided in Table 26-G were based on the following safety factors:

- FS = 8 for uninspected construction and
- FS = 4 for inspected construction.

As with other materials, for wind or seismic loading allowable loads could be increased by one-third.

The determination of anchor allowable capacities from tests was typically based on a sample size of three. Tests were usually conducted in three concrete compressive strengths (2, 4, and 6 ksi) and results were aggregated irrespective of failure mode.

In the 1980s, the 45-degree cone method was promoted rather than competing formulations based on, for example, plasticity models (Shipp and Haninger, 1983). The basic idea was that a uniform tensile stress of $4\sqrt{f_c'}$ could be assumed to exist over the theoretical 45-degree conical failure surface at the moment of rupture. Integration of the vertical component of stress over the failure surface resulted in an expression that predicted the tensile capacity to be related to the square of the embedment depth. A review with a database consisting of thousands of individual tests proved this to be unconservative for the typical range of embedment depths (7 to 9 anchor diameters) used in construction. The results of this review were published in "RB.13-Comparison of Concrete Capacity Design Method and ACI 349-97" provided in Appendix B of ACI 349R, *Commentary to Code Requirements for Nuclear Safety Related Concrete Structures* (ACI, 2001). Anchors are often used in groups, and the 45-degree cone method predicts full capacity for anchors in a group if they are spaced at twice the embedment depth and are at least one times the embedment depth from any edge. Subsequent testing showed this to be particularly unconservative.

In the 1990s, a concerted effort was made to shift anchor design in the code from allowable stress design to strength design (known at the time as load and resistance factor design), along the way bringing in concepts such as the 5% fractile of anchor resistance (to account for scatter), the influence of concrete cracking on anchor resistance, and the concrete capacity design methodology that had been developed by W. Fuchs and J. Breen at the University of Texas at Austin. These concepts had been developed over the previous decade under the leadership of Rolf Eligehausen at the Institut fuer Werkstoffe im Bauwesen, University of Stuttgart (CEB, 1991). While this represented a substantial improvement in the equations used to predict concrete failure modes, the traditional global safety factors applied to anchors for static loading were essentially preserved.

For an average load factor of 1.55 (at the time, ACI load factors were 1.4 for dead load and 1.7 for live load), the global safety factor on the mean capacity provided by the ACI 318 strength reduction factors was as follows:

- 2.8 for cast-in-place anchors;
- 3.2 for Category 1 (superior reliability) post-installed anchors;
- 3.8 for Category 2 (typical reliability) post-installed anchors; and
- 4.6 for Category 3 (lower reliability) post-installed anchors.

Thus, a Category 2 anchor, which was assumed to represent the typical level of reliability, carried a global safety factor close to the traditional value of 4.

For seismic loads, however, the situation is quite different. The one-third stress increase factor that lowered the global safety factor to 3 for inspected anchors developed from the assumption that there is a low probability of loading simultaneity (Mueller and Carter, 2003). With the shift to load and resistance factor design, however, this low probability of loading simultaneity is taken into account in the load factors. More importantly, for post-installed anchors, seismic loading implies a host of other conditions that may negatively impact anchor strength. These include damage to the concrete around the anchor (this affects cast-in anchors equally), fluctuations in the drilled hole diameter due to crack cycling, and uneven load distribution on anchor groups resulting from non-uniform yielding patterns and anchor stiffness at loads exceeding the anchor preload.

The original formulation of the strength design provisions for anchors in the 2000 IBC, that were borrowed with permission from the soon to be published ACI 318-05 provisions, required that anchor designs be controlled by the strength of the steel anchor element (bolt) or for the yield strength of the anchorage. The requirement that the anchor steel strength control the anchor design derives from provisions developed for nuclear construction in ACI 349R-01. The general concept was that steel failure is a more predictable and stable failure mode and provides ductility in the connection. In addition, a reduction factor of 0.75 was applied to the anchor resistance in tension and shear, regardless of the failure mode. The global safety factor on the mean resistance for the seismic case under the rules established by ACI for a post-installed anchor with superior reliability is 5.

So far, only anchors that meet the ductility criteria (e.g., yielding anchors) have been addressed. In 2008, responding to the imposition of ad hoc load increase factors in the *NEHRP Provisions* and the IBC for so-called "non-ductile anchors" and the lack of this option in the original Appendix D in ACI 318-05, ACI imposed its own penalty on the resistance side for anchors that did not meet ductility criteria. The adopted factor of 0.4 was the inverse of the 2.5 factor established in the *NEHRP Provisions*. It can also be compared to the penalty established in ASCE/SEI 7-05 for anchors that either were not prequalified or were not designed in accordance with ACI 318-05 ductile failure provisions (i.e., taking the 1.3 load increase factor together with the $R_{p,\max}$ value of 1.5). For a component assigned an R_p of 3, an increase of roughly the same magnitude ($1.3 \times 3 / 1.5 = 2.6$) was implied.

Regardless of the origin, application of an additional reduction factor of 0.4 increases the global safety factor a post-installed anchor with superior reliability to 12.5 on the mean static resistance in uncracked concrete, compared with 3 under allowable stress design. Note that this level of safety has essentially been in place for the past 15 years. The 2.5 factor for "non-ductile anchorage" first appeared in the 2000 *NEHRP Provisions*, in parallel with the initial appearance, in the 2000 IBC, of anchor strength design provisions based on the concrete capacity method

The existence of multiple and sometimes redundant paths in the building code and reference standards for the treatment of anchors that did not otherwise satisfy ductility criteria led to substantial confusion. Consequently, in ASCE/SEI 7-10 including Supplement No. 1, both the 1.3 factor and the R_p penalty were replaced with the limit of 6 on R_p for anchor calculations as previously discussed. At the same time, ongoing changes were made in the ACI provisions for seismic design of anchors. With the publication of ACI 318-11, the 0.75 factor as applied to the calculation for steel resistance and edge breakout in shear had been eliminated. The check for anchor yielding in tension was amended to require that 120% of the nominal (as opposed to factored) anchor steel strength be less than the nominal concrete breakout strength (a separate check against the factored load that includes ϕ and the 0.75 factor is required) and a stretch length requirement was added to ensure a reasonable degree of deformation capacity. An observational basis for the stretch length requirement in ACI 318-11 is provided by Soules, Bachman, and Silva (2016). Most significantly, the 0.4 factor for non-ductile anchors was replaced with a direct reference to Ω_0, a change which has important ramifications for overturning calculations on base-mounted equipment (Johnson et al., 2016). This closely reflects the approach taken in the 2000 *NEHRP Provisions*:

> "9.2.3.3.3: In structures assigned to Seismic Design Categories C, D, E, or F, anchors shall be designed to be governed by tensile or shear strength of a ductile steel element unless Section 9.2.3.3.4 is satisfied."

“9.2.3.3.4: In lieu of Section 9.2.3.3.3, the attachment that the anchor is connecting to the structure shall be designed so that the member being attached will undergo ductile yielding at a load level no greater than 75 percent of the minimum anchor design strength or the minimum anchor design strength is at least Ω_0 times the attachment force determined from design loads of the attached structure or 2.5 times the attachment force determined from the design loads of the attached nonstructural component.”

Nevertheless, while useful for establishing a clear path to determining anchor loading and resistance and for clarifying the role of the various factors used for seismic design of anchors, these changes did little to alter the global safety factor that was put in place with the introduction of strength design in 2000. Although research continues into the appropriate level of overstrength required where ductility is not provided, the current provisions should not necessarily be assumed to result in over-design of anchors to resist seismic loads. This is particularly true in the case of equipment, piping, and architectural components (ceilings, lighting, etc.) anchored to slabs and walls that also function as diaphragms and vertical shear-resisting elements subject to increased levels of cracking than is anticipated in current qualification standards. It is clear, however, that the current safety levels for anchorage are significantly greater than those that were commonly in use before the turn of the century.

3.6 Seismic Provisions for Nonstructural Components in Building Codes in Other Countries

Building codes in other countries with high seismicity have been addressing the seismic design of nonstructural components in a manner similar to United States building codes. The overarching similarity between building codes in all countries is that the simplified formulae for the static design force of acceleration-sensitive nonstructural components generally consist of the following parameters: (1) design peak ground acceleration (PGA); (2) a factor to address the amplification of peak floor acceleration (PFA) over the building height relative to the PGA; (3) a factor to address the amplification of the response at the center of mass of the nonstructural component relative to the PFA; (4) a factor to address the acceptable nonlinear response (ductility) of the component; and (5) an importance factor. Building codes differ on their assumptions for each of these five parameters, and their minimum or maximum values for the design forces. The following sections highlight differences in building codes from Europe, Japan, New Zealand, Chile, and Canada.

3.6.1 Building Codes in Europe, Japan, New Zealand, Chile, and Canada

EuroCode 8, *Design of Structures for Earthquake Resistance – Part 1: General rules, seismic actions and rules for buildings* (ECS, 2004), has a “Seismic Coefficient” that addresses both amplification of peak floor acceleration over the building height and the component amplification factor. The "Seismic Coefficient" assumes a linear variation over the building height (increases from 1 at the base to 2 at the roof level), and has a nonlinear component (for the component amplification factor) that depends on ratio of the periods of the nonstructural component and building.

The Japanese guideline, *Building Equipment Seismic Design and Construction Guidelines* (BCJ, 2007), has a factor for the PFA to PGA ratio that can take three different values: 1, 1.5, or 2.5 at the base, mid height, and roof of the building, respectively. The component amplification factor in the Japan building code has two different values: 2.0 for vibration-isolated equipment, and 1.5 for other equipment.

The New Zealand standard NZ1170.5, *Structural Design Actions - Part 5: Earthquake Actions - New Zealand* (SNZ, 2004), has a "Height Coefficient" that increases linearly from 1 at the building base to 3 at 20% of the building height, and equals 3 for all levels higher than 20% of the building height. The code has a component amplification factor that depends on the nonstructural component period (independent of the building period), and varies between 0.5 (for components with a period shorter than 0.75 seconds) and 2.0 (for components with a period longer than 1.5 seconds).

The Chilean code NCh 2369.Of2003, *Earthquake Resistant Design of Industrial Structures and Facilities* (INN, 2003), has a linear factor for the PFA to PGA ratio that increases from 1 at the building base to 4 at the roof level. The component amplification factor in NCh 2369 can take a maximum value of 2.2 and is a nonlinear function of the ratio between the periods of nonstructural component and building.

The Canadian standard CSA S832-06, *Seismic Risk Reduction of Operational and Functional Components (OFCs) of Buildings* (CSA, 2006), has a linear factor for the PFA to PGA ratio that is similar to that of ASCE/SEI 7 (i.e., it increases from 1 at the building base to 3 at the roof level). The component amplification factor in this standard depends on the ratio between the periods of the nonstructural component and building, and varies between 1 and 2.5.

3.6.2 Seismic Provisions for Nonstructural Components in ISO 13033:2013

The International Organization for Standards (ISO) standard 13033:2013, *Basis for Design of Structures--Loads, Forces and Other Actions--Seismic Actions on Nonstructural Components for Building Applications* (ISO, 2013), was published to establish the means to derive seismic actions on nonstructural components and systems. ISO 13033 is not a legally binding and enforceable code, and does not specifically cover industrial facilities (including nuclear power plants), but it is nonetheless a source document that can be utilized in the development of codes of practice by the competent authority responsible for issuing structural design regulations.

ISO 13033 recommends a factor for the ratio of PFA and PGA that varies linearly over the building height. Unlike ASCE/SEI 7, ISO 13033 recommends the rate of increase of PFA to PGA ratio over the building height depends on lateral force-resisting system of the building (e.g., different formula for steel moment frames and braced frames). ISO 13033 assigns a cap of 3.5 to the PFA to PGA ratio at the roof level for all lateral force-resisting systems.

ISO 13033 categorizes the nonstructural components as rigid or flexible depending on the ratio of the periods of the nonstructural component and building. Component amplification factors are assigned based on this categorization and their support condition.

The differences of the building codes that were covered in the previous section and ISO 13033 from standpoints of the PFA/PGA ratio and component amplification factor are summarized in Table 3-4.

Table 3-4 Summary of Methodology for Seismic Nonstructural Design

International Standard or Code	Peak Floor Acceleration			Component Amplification Factor is Period Dependent
	Linear Height Transition	*Building Period Dependent*	*Maximum PFA/PGA Ratio at Roof*	
ISO 13033	Yes	Yes	3.5	Yes
Canada	Yes	No	3	Yes
Chile	Yes	No	4	Yes
EuroCode 8	Yes	No	2	Yes
Japan	No	No	2.5	No
New Zealand	No	No	3	Yes

The design recommendations contained in ISO 13033 are based on achieving the following performance objectives:

- to prevent human casualties associated with falling hazards and blockage of egress paths;
- to ensure post-earthquake continuity of life-safety functions within the building (e.g., sprinkler piping);
- to ensure continued post-earthquake operation of essential facilities (e.g., hospitals, fire stations);
- to maintain containment of hazardous materials; and
- to minimize damage to property.

To achieve the seismic design objectives, ISO 13033 establishes the following basic performance criteria:

- Nonstructural components and systems subjected to the severe earthquake ground motions that are specified at the building site (ultimate limit state: ULS) should be designed, qualified by testing or qualified by experience data to demonstrate that the nonstructural components and systems: (1) will not collapse, detach from the building structure, overturn or experience other forms of structural failure, breakage or excessive displacement (sliding or swinging) that could cause a life safety hazard; (2) will perform as required to maintain continuity of life safety functions (e.g., fire-fighting systems, elevators, and other similar vital life safety systems); (3) will remain leak tight as required to prevent unacceptable release of hazardous materials (e.g., vessels, tanks, and piping and gas circulation systems that contain hazardous materials); and (4) will operate as necessary immediately following the earthquake event to ensure continued post-earthquake function of essential facilities.
- Nonstructural components and systems subjected to the moderate earthquake ground motions specified at the building site (serviceability limit state: SLS), will perform within accepted limits including limitation of financial loss.

3.7 Enforcement of Nonstructural Component Design Provisions

Although nonstructural seismic provisions have been incorporated into the building code for decades, proper implementation of the provisions in construction practice has been elusive. Except in areas subject to frequent earthquakes, many design professionals have limited familiarity with seismic design of nonstructural components. Responsibility for performing seismic design of nonstructural components often rests with subcontractors rather than the project engineer, who, in some cases, has no involvement at all in the nonstructural design process. Many nonstructural items are commonly excluded from construction drawings and identified in the project specifications as provided by the contractor on a "design-build" basis. Other items are identified as "owner furnished and installed." In these cases, the installation details may not be submitted to the design engineer or the building department for review.

Few building departments have resources devoted to plan review of construction documents for nonstructural seismic bracing. Field enforcement of nonstructural seismic requirements is often lacking because details associated with seismic restraint of nonstructural components are not on the approved drawings. Design professionals could perform field observation of nonstructural component installations, but this task is often not included in their scope of work. In many cases nonstructural components are installed after occupancy of the building and, in some cases, over the life of the building as new technology is developed or new tenants occupy the space.

Most items classified as contents or furniture are specifically exempted from the seismic provisions of ASCE/SEI 7-16. Although not regulated by the building code, these items can pose a risk to safety and continuity of operations in the event of an earthquake.

3.8 Evaluation Methodologies for Nonstructural Components in ASCE/SEI 41-13

Performance-based requirements for existing buildings are found in ASCE/SEI 41-13, *Seismic Evaluation and Retrofit of Existing Buildings*. ASCE/SEI 41-13 includes a performance-based approach for the evaluation and retrofit of nonstructural components. It offers procedures for assessing the condition of existing nonstructural components, evaluating their expected seismic performance, selecting seismic retrofit objectives, and identifying potential interactions between structural and nonstructural components.

ASCE/SEI 41-13 permits the user to select from among three different nonstructural performance levels: Life Safety, Position Retention, and Operational, and offers a range of ground shaking hazard levels for use in evaluation and retrofit design. Nonstructural performance levels and illustrative damage descriptions for two nonstructural components are illustrated in Table 3-5. Unlike ASCE/SEI 7-16, in which nonstructural performance is tied to a single ground shaking level, nonstructural performance levels in ASCE/SEI 41-13 can be linked to any selected seismic hazard level to generate a nonstructural performance objective. For nonstructural performance objectives that are comparable to ASCE/SEI 7-16, the BSE-1N seismic hazard level of ASCE/SEI 41-13 is used (taken as two-thirds of the MCE_R hazard level). When evaluating existing buildings, reduced hazard levels are used, and the BSE-1E seismic hazard level (ground shaking with a 20% probability of exceedance in 50 years) is typically selected. Depending on the location, the BSE-1E seismic hazard level may produce substantially lower shaking intensity (and design forces) than the BSE-1N or the ASCE/SEI 7-16 design earthquake level. A Basic Performance Objectives for new and existing buildings are established based on a combination of structural and nonstructural performance levels, in combination with seismic hazard levels, and the combinations vary based on the building Risk Category.

Nonstructural performance objectives are expressed in terms found in ASCE/SEI 7-16, but the meanings are different. For example, at the Life Safety Nonstructural Performance Level in ASCE/SEI 41-13, some components may displace or topple, and only items judged to be capable of causing serious injury or death are anchored and braced. This performance level is substantially lower than the concept of life safety in ASCE/SEI 7-16. The Position Retention Nonstructural Performance Level in ASCE/SEI 41-13, combined with the BSE-1N seismic hazard level, provides similar, but not identical performance as ASCE/SEI 7-16 for ordinary occupancies. Acceptance criteria for the Operational Nonstructural Performance Level, which is associated with Risk Category IV buildings (e.g., hospitals), essentially mirror the nonstructural seismic provisions in ASCE/SEI 7-10.

Table 3-5 Nonstructural Performance Levels and Illustrative Damage of Architectural Components (excerpt from Table C2-5 of ASCE/SEI 41-13)

	Nonstructural Performance Levels		
Component	*Life Safety (N-C)*	*Position Retention (N-B)*	*Operational (N-A)*
Cladding	Extensive distortion in connections and damage to cladding components, including loss of weather-tightness and security. Overhead panels do not fall.	Connections yield; minor cracks or bending in cladding. Limited loss of weather-tightness.	Connections yield; negiligble damage to panels. No loss of function or weather-tightness.
Glazing	Extensively cracked glass with potential loss of weather-tightness and security. Overhead panes do not shatter or fall.	Some cracked panes; none broken. Limited loss of weather-tightness.	No cracked or broken panes.

The analytical approach for determining design acceleration and displacement demands are similar to those in ASCE/SEI 7-10. Components are classified as being primarily acceleration sesnsitive or displacement sensitive. In some cases, components that are displacement sensitive may not require a force analysis. For come components, prescriptive procedures found in approved codes and standards may be utilized in lieu of performing a seismic analysis. Where analysis is required, the default force and deformation equations are essentially identical to those found in ASCE/SEI 7-10.

Chapter 4

Standards for Nonstructural Seismic Protection

Design and installation of some nonstructural components and systems are governed by national standards that are developed by independent organizations and adopted by reference into ASCE/SEI 7 or the building code. Some standards are incorporated with amendments, and there is a strict precedence followed when conflicts arise between the building code and standards. Where differences occur, the provisions of the building code apply. Similarly, amendments to standards that are incorporated by reference in ASCE/SEI 7 take precedence over the provisions of the standard. Although there are groups that develop and incorporate seismic design provisions in their standards, many significant categories of nonstructural components lack groups that champion development of seismic design provisions. For example, the design focus for heating, venting and air-conditioning (HVAC) components is typically on operational performance, emphasizing goals such as energy efficiency, durability, and economy of operation, while no national standard has been developed that includes a focus on seismic performance. In this chapter, some of the common standards that address seismic protection of nonstructural components and systems are described.

4.1 NFPA 13, *Standard for the Installation of Sprinkler Systems*

Fire protection sprinklers are designed and installed in accordance with the National Fire Protection Association (NFPA) standard NFPA 13, *Standard for the Installation of Sprinkler Systems* (NFPA, 2016). The seismic provisions for fire sprinkler systems were developed to allow simplified seismic design procedures that would produce a system in compliance with the requirements of ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010). NFPA 13 uses a simplified seismic factor, C_p, which considers ground shaking, dynamic amplification, component response, and location in the building. Conservative assumptions are made for each variable, so that the only information needed to find C_p is the short-period mapped spectral acceleration for the Maximum Considered Earthquake (MCE), S_s. The methodology was developed to include concepts familiar to the fire sprinkler industry, including the use of "zone of influence" (ZOI) to determine brace spacing and force demands to bracing supports and attachments. The spacing and configuration of bracing can be constrained by both the capacity of the support elements (braces and fittings) and by the flexural capacity of the pipe. In lieu of the simplified procedures in NFPA 13, the piping system can be designed in accordance with the provisions of Chapter 13 in ASCE/SEI 7-10.

Seismic design of the sprinkler system begins with determination of the value of C_p, which is a short-period mapped spectral acceleration. A preliminary bracing arrangement is then developed, subject to prescriptive limits on brace spacing and arrangement for both mains and branch lines. For given brace location, the designer than calculates the weight of piping in the ZOI, which includes the weight of the water-filled branch lines and the tributary weight of the main lines. Multiplying this value by the seismic factor, C_p, provides a point load, F_p. This point load is used to determine whether the given spacing between lateral supports is acceptable. A group of tables based on piping materials and connection type is provided. The user compares the force demand, F_p, against the maximum permitted ZOI load which is a function of pipe diameter and distance between bracing points. If the permitted ZOI load is greater than F_p, the brace spacing is acceptable (pipe stresses are within the acceptable range) and the design proceeds to the next step. If not, the brace spacing is modified and F_p is recalculated. Once the brace spaces are acceptable, the brace capacities are checked, by comparing F_p to tabulated capacities of different bracing configurations. Finally, attachments to the structure through the use of expansion-type anchors to concrete or masonry or connectors to wood are selected, again by comparing F_p to tabulated capacities for different types of attachments and brace configurations. A full description of the seismic design procedure development with examples can be found in Annex E of the 2016 NFPA 13.

In addition to the bracing calculations, fire sprinkler piping must meet prescriptive requirements for system configuration including component listings, clearances, and limitations on brace spacing and hanger installation.

Lateral bracing requirements for fire sprinkler systems have been in the installation standards since 1947. The effectiveness of the fire sprinkler bracing provisions was noted following the 1964 Great Alaska earthquake. Where damage was observed to fire sprinkler systems, it was usually believed to be a consequence of the failure of other building elements. In subsequent earthquakes, much of the damage to fire sprinkler systems was the result of failures at threaded connections due to movement of unrestrained branch lines, failure of power actuated fasteners in concrete, and the interaction of sprinkler sprigs, drops, and heads with ceilings and adjacent components. Sprinkler heads are especially vulnerable to impact, and even a single damaged sprinkler head can result in significant water damage.

Revisions to the sprinkler system hanging and bracing requirements over the years have addressed many of these issues. Earthquake damage including sprinkler failures continues to be reported, and because seismic requirements for these systems have been in place for so long the damage is noted to occur in “braced systems.” Given that the age of the system installation is almost never reported, it is difficult to gauge the effectiveness of the current sprinkler design provisions based on performance in recent earthquakes. It is clear that avoiding interaction between the sprinkler system and adjacent building elements is a key to better performance.

4.2 ASME A17.1, *Seismic Provisions for Elevators*

Elevators are among the most important mechanical systems whose seismic failure during an earthquake can cause life safety concerns and result in interruption of service (in addition to direct loss associated with the repair). The seismic response of elevators is affected by both seismic acceleration and drift responses in building structures.

Seismic design of elevators, like other building nonstructural components, is currently per Chapter 13 of ASCE/SEI 7-10. Section 13.6.10 of ASCE/SEI 7-10 has adopted seismic requirements of the American Society of Mechanical Engineers (ASME) document ASME A17.1, *Safety Code for Elevators and Escalators* (ASME, 2010). From the ASME A17.1 standpoint, there are three levels of seismicity defined by the following: Seismic Zone 1; Seismic Zone 2; and greater than Seismic Zone 2. Elevators in a Seismic Zone 1 are exempted from the seismic requirements of Section 8.4 in ASME A17.1.

ASME A17.1 provides design formula and construction requirements such as horizontal clearances without providing commentary for the theoretical background. The code requires the seismic design consider simultaneous action of horizontal and vertical earthquake forces. Design forces depend on the seismic zone and are always expressed as a ratio of the seismic weight. Elevators in a greater than Seismic Zone 2 are subjected to seismic requirements and loads that are twice as stringent or larger than those for elevators in a Seismic Zone 2. The code assigns a factor of 2 to calculate impact loads from static design loads. The design seismic weight includes the self-weight plus 40% of the elevator capacity. Maximum combined stresses in fastenings and their parts due to the specified seismic forces are limited to 88% of the yield strength of the material used. The code requires information regarding horizontal seismic forces be displayed on elevator layout drawings. The code requires that the guide rails of elevators be constructed of T-sections with requirements for minimum moment of inertia along two principal axes. To avoid any damage under a seismic acceleration of up to 0.5g, the code provides minimum bracket spacing, and maximum allowable weight for the guard rails.

Under a series of conditions illustrated in a flowchart diagram in ASME A17.1 (e.g., if a traction elevator operates at a speed of 150 feet per minute or higher), the code requires two “fail-safe” earthquake protective devices: (1) a seismic switch; and (2) a displacement switch. A seismic switch is intended to provide a signal when shaking stronger than a defined threshold begins, and, subsequently, the cars in motion must stop at the nearest floor, open their doors, and shut down. A displacement switch, on the other hand, is intended to detect when the counterweight is moving outside its normal operating plane of travel or has left its guide rails. Once the displacement switch is activated, the cars in motion must stop and then proceed to the nearest landing at a reduced speed in the direction away from the counterweight. The implementation of seismic and displacement switches has practical implications, however, as these switches can be activated by non-seismic shaking induced by vibrating machinery, nearby traffic or construction work (Suarez and Singh, 2000).

Seismic design of elevators in the United States seems to have achieved the life safety objective so far as seismic damage to elevators has not been directly associated with injuries or fatalities during the past earthquakes in the United States (Suarez and Singh, 2000). However, seismic damage to elevators that has hampered continued service of important facilities has been reported after almost every major earthquake (e.g., 674 cases of derailment of elevator counterweights were reported after the 1971 San Fernando earthquake (Ayres and Sun, 1973) and and 688 cases after the 1994 Northridge earthquake, (McTiernan, 1994)). Suarez and Singh (2000) studied 19 cases of elevator damage occurring during earthquakes from the 1964 Great Alaska earthquake to 1995 Kobe earthquake. They summarized the recurrent observed damage as the following: (1) permanent deformation of guard rails and/or damage to their anchorage; (2) derailment of counterweights from their guardrails and subsequent damage due to impact between loose counterweights and passenger cars; (3) excessive displacement or toppling of control panels, traction machines, motor generators; and (4) damage to ropes by projections or protuberances in the hoist ways, or jumping from drive. Suarez and Singh reported that in addition to the observed damage, in several cases seismic switches failed to trigger properly and promptly.

The design accelerations used for elevators, limited to 0.5g, are no longer compatible with the latest estimates of earthquake shaking potential, which predict substantially stronger shaking. The observed damages to the elevators during the past earthquakes correlate with findings of several research studies that design loads prescribed by the ASME 17.1 can be significantly unconservative (500 to 650% for low-rise buildings and 50 to 250% for tall buildings located in a Seismic Zone 4 in the United States (Rutenberg et al., 1996; Segal et al., 1994; 1995; 1996; and Levy et al., 1996).

The seismic design procedures for elevators are prescriptive, and it is therefore difficult to compare the elevator design procedures to those for other nonstructural components. Elevator design forces are not compatible with those expected using the latest hazard maps and design procedures for other nonstructural components. Because proper operation of traction elevators requires maintaining close alignments of the car and counterweights, designing traction elevators for operation following a design earthquake is challenging. Hydraulic elevators have been shown to be less vulnerable to seismic damage. Both traction and hydraulic elevators can be rendered inoperable if building story drift results in racking of the doors.

4.3 ASTM E580, *Standard Practice for Installation of Ceiling Suspension Systems for Acoustical Tile and Lay-in Panels in Areas Subject to Earthquake Ground Motions*

Suspended acoustic lay-in tile ceilings systems are generally installed in accordance with the prescriptive requirements of American Society of Testing and Materials (ASTM) standard ASTM E580, *Standard Practice for Installation of Ceiling Suspension Systems for Acoustical Tile and Lay-in Panels in Areas Subject to Earthquake Ground Motions* (ASTM, 2014). This standard incorporates the latest information generated by full scale shake table testing of ceiling systems. The seismic requirements of ASTM E580 are prescriptive and, provided that the suspended ceiling meets the configuration and weight criteria, no engineering calculations are needed. Suspended ceilings requirements vary by seismic hazard, as expressed by the Seismic Design Category (SDC) of the structure, and the size of the suspended ceiling defined by area. The standard specifies a maximum weight of the ceiling, including the weight of light fixtures and HVAC air terminals that are supported on the ceiling grid. If the limit is exceeded, then the ceiling must be designed using the provisions of Chapter 13 of ASCE/SEI 7-10.

There are no seismic requirements for ceilings installed in SDC A and SDC B, low seismic hazards areas. Ceilings in these areas are not subject to the requirements of ASTM E580. In areas of moderate risk, defined as SDC C, the objective of the standard is to provide an unrestrained ceiling that accommodates the movement of the structure during an earthquake. The main runners and cross-runners of the ceiling grid and their splices and connections must meet minimum ultimate test load requirements. The spacing of hangers and the size of the perimeter closure angles is also specified. Unless independently supported, all light fixtures must by positively attached to the ceiling system and safety wires provided, to prevent the light fixture from falling should it detach from the ceiling grid.

For ceilings in areas of high seismic risk, defined as SDC D, SDC E, and SDC F, the objective of the standard is to provide a restrained ceiling that involves connections to the perimeter walls and either rigid or non-rigid bracing assemblies. The main runners of the ceiling grid must be heavy-duty and main runner and cross-runner splices and connections must meet minimum ultimate test load requirements that are several times higher than those for ceilings in SDC C. Perimeter closure angles must provide a 2-inch minimum support ledge, and there are minimum edge clearances requirements to allow for axial movement of the runners. Clips at the perimeter of the grid may allow a smaller closure angle will be permitted in ASCE/SEI 7-16, but must meet detailing provisions that include the use of screws with the clips to prevent loss of vertical support. Lateral bracing assemblies are required for all ceiling areas greater than 1,000 square feet. These assemblies may consist of rigid braces or splay wires with compression posts. Ceiling areas greater than 2,500 square feet must have seismic separation joints that break the ceiling into areas smaller than 2,500 square feet. Clearances must be provided around sprinkler heads and other penetrations of the ceiling, unless flexible sprinkler hose devices are used. All light fixtures must be positively attached to the ceiling system and safety wires provided, to prevent the light fixture from falling should it detach from the ceiling grid. Fixtures weighing more than 56 pounds must be independently supported.

Suspended acoustical tile ceilings have been the subject of several major shake table testing programs. These tests show that when designed and installed in accordance with the current industry standards, suspended acoustical tile ceilings up to 1,000 square feet are expected to perform well in the design earthquake. Some damage to ceiling tiles due to interaction with sprinkler heads was observed in the shake table testing and minor damage to the grid may occur. Ceilings with areas larger than 1,000 square feet have not been tested. Shake table test have been performed on ceilings that are rectangular or square in plan. The performance of ceilings in irregularly shaped areas has not been tested.

Enforcement of the ceiling standard is sometimes problematic, even in new construction. Because the suspended ceiling system is the often installed last, locating splay wire and compression post assemblies can be difficult without careful coordination of the various trades. The standard specifies that when ceiling areas exceed 2,500 square feet, it must have seismic separation joints that break the ceiling into smaller areas, but this does not always occur in practice.

4.4 RMI ANSI MH16.1, *Specifications for the Design, Testing and Utilization of Industrial Steel Storage Racks*

Past earthquakes have repeatedly shown that seismic damage to storage racks can have staggering life safety consequences, and/or result in direct loss and interruption of service. The following is a brief summary of how seismic codes and guidelines for steel storage racks have evolved to address the lessons learned from past earthquakes and research (more information is available in FEMA 460, *Seismic Considerations for Steel Storage Racks Located in Areas Accessible to the Public* (FEMA, 2005)).

The first set of any engineering design requirements for steel storage racks, *Minimum Engineering Standards for Industrial Storage Racks*, was published in 1964 by Rack Manufacturers Institute (RMI, 1964). The seismic design requirements, which were similar to those prescribed by the *Uniform Building Code* for the building structures, appeared in the 1972 *Interim Specifications for the Design, Testing and Utilization of Industrial Steel Storage Racks* (RMI, 1972). The seismic design of steel storage racks along the down-aisle and cross-aisle directions per 1972 RMI *Interim Specifications* would be similar to the seismic design of ordinary moment frame and braced framed structures, respectively.

The 1973 *Uniform Building Code* (ICBO, 1973) refers storage racks in the form of a footnote to a list of structures. The 1976 *Uniform Building Code* (ICBO, 1976) references UBC Standard 27-11 for seismic design of steel storage racks. UBC Standard 27-11 is essentially a direct reference to a standard produced by the Rack Manufacturers Institute, Inc. (RMI, 1973).

NEHRP introduced design values for storage racks for the first time in the 1991 edition. The recommended design seismic forces were for an allowable stress design method and were calculated independent of the rack period or seismic force-resisting system. In the 1994 *NEHRP Recommended Seismic Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1995), design parameters for storage racks including *R* values along two principal axes and an importance factor were prescribed to yield design loads that were more consistent with the RMI seismic design criteria.

ASCE/SEI 7 has addressed the seismic design of steel storage racks as non-building structures since the 1995 edition (based on 1994 *NEHRP Provisions*). ASCE/SEI 7-10 addresses the seismic design requirements for steel storage racks in Chapter 15 (Seismic Design Requirements for Nonbuilding Structures). Section 15.5.3 of ASCE/SEI 7-10 requires steel storage racks at or below grade be designed in accordance with American National Standards Institute, ANSI MH16.1, *Specifications for the Design, Testing and Utilization of Industrial Steel Storage Racks* (RMI, 2012), and use the force and displacement requirements, with exceptions for the minimum design force for above-grade elevation racks and design requirements for base plates and shims. The minimum design force for above-grade elevation racks is per the formula of Chapter 13 of ASCE/SEI 7 for acceleration-sensitive nonstructural components. As an alternative to ANSI MH16.1 (modified per Section 15.5.3.1 through Section 15.5.3.3), ASCE/SEI 7-10 allows design in accordance with the requirements of Section 15.1, Section 15.2, Section 15.3, Section 15.5.1, and Section 15.5.3.5 through Section 15.5.3.8.

Section 2.6 of ANSI MH16.1 covers the design earthquake loads. Storage racks that are shorter than 8 feet in height to the top load shelf, or are connected to buildings or other structures, are exempted from the seismic design per ANSI MH16.1. The standard has requirements for the minimum seismic forces, connection rotation capacity, and clearance between the storage rack and the building or other structures to avoid damaging contact during an earthquake. Unless used to store hazardous material, the standard assigns an Occupancy Category II structures to storage racks.

Steel storage racks have performed poorly in some earthquakes. Because the rack systems are designed to be reconfigured and relocated, the connection designs must balance ease of assembly and disassembly with strength, stiffness and ductility. Seismic performance is highly dependent on proper installation and maintenance, and the ease with which the rack systems can be assembled and disassembled presents on ongoing challenge. Racks are installed in areas subject to forklift traffic, and impact damage to columns in the rack system occurs, especially at corner columns. Damage to the rack columns can substantially degrade seismic performance. While many jurisdictions require building permits for initial rack installation, the rack system may be subsequently modified. Bracing elements and anchor bolts may be omitted, substantially degrading seismic performance. There is a large market for used rack systems, and it is possible to inadvertently install a system with low seismic capacity in a high seismic area.

4.5 ASME B31.1, *Power Piping*

In the context of seismic design of building nonstructural components, piping systems are categorized as mechanical components. Section 13.6.8.1 of ASCE/SEI 7-10 states that “pressure piping systems, including their supports, designed and constructed in accordance with ASME B31 shall be deemed to meet the force, displacement, and other requirements of this section. In lieu of specific force and displacement requirements provided in ASME B31, the force and displacement requirements of Section 13.3 shall be used. Materials meeting the toughness requirements of ASME B31 shall be considered high-deformability materials. This Standard establishes a method for the seismic design of above-ground piping systems in the scope of the ASME B31, *Code for Pressure Piping*.”

ASME B31.1, *Power Piping* (ASME, 2014) uses the allowable stress design method, and has prescriptions for a designer on how to consider combined effects of operational loads and seismic forces induced by inertial effects as well as deflections, while maintaining the pressure integrity under high operating temperatures and pressures. ASME B31.1 provides allowable stresses for load combinations that include “occasional loads,” namely, earthquake and wind loads. In 2008, to provide more explicit and structured guidance for seismic design of new piping systems, as well as retrofit of existing systems, ASME published ASME B31E, *Standard for Seismic Design and Retrofit of Above-Ground Piping Systems* (ASME, 2008).

ASME B31E defines its scope as it “applies to above-ground, metallic piping systems in the scope of the ASME B31 *Code for Pressure Piping* (B31.1, B31.3, B31.4, B31.5, B31.8, B31.9, and B31.11). The requirements described in this Standard are valid when the piping system complies with the materials, design, fabrication, examination, testing, and inspection requirements of the applicable ASME B31 code section."

ASME B31E requires triaxial seismic loading in the form of horizontal and vertical seismic static loads, or horizontal and vertical seismic loads resulted from response spectrum analyses. In either case, seismic input is to be specified in accordance with Chapter 13 in ASCE/SEI 7-10 or site-specific spectrum analyses.

As for analysis methods, depending on: (1) the classification of the piping system (critical or noncritical); (2) the magnitude of the seismic input; and (3) the pipe size, the standard requires design "by rule" or "by analysis." In situations allowing design by rule is permitted, the seismic design of piping systems may be achieved by providing lateral seismic restraints at spacing equal or smaller than a maximum spacing given by the formula (or tabulated information) in the standard. In situations where design by analysis is required or where it is applied by the designer as an alternative method, the calculated elastic longitudinal stresses due to the design earthquake loads (calculated by static or dynamic analysis) shall comply with the stress interaction formula in the standard. In addition to the two prescribed methods, the standard allows more detailed analysis techniques, including fatigue, plastic, or limit load analysis.

The ASME B31 also has provisions for the mechanical joint, seismic restraints, and mechanical equipment connected to the piping system, and without providing details it mentions that "Piping systems shall be evaluated for seismic interactions. Credible and significant interactions shall be identified and resolved by analysis, testing, or hardware modification."

Experience in past earthquakes has shown that piping systems installed in accordance with ASME B31 perform well. The operational requirements such as pressure and thermal expansion produce designs that are robust. While common in industrial settings, many piping systems found in commercial and institutional settings are not required to comply with ASME B31.

Chapter 5

Practical Guidelines and Performance-Based Design Tools

Reducing nonstructural damage in future earthquakes requires awareness of potential nonstructural risks, understanding the costly consequences of nonstructural failures, and access to tools designed to limit future losses. Effective seismic risk reduction strategies cultivate an understanding of nonstructural issues among a broad population, including those with direct responsibility for design and/or installation. Several practical guidelines have been published to increase the likelihood that nonstructural components and systems will be installed or retrofitted with effective and code-compliant seismic restraints. Some guidelines have been developed for non-technical audiences with direct or indirect responsibility for overseeing construction or facilities maintenance.

Concurrent with the development of practical guidelines, the engineering community has been advancing performance-based design methodologies. These methodologies can be used to estimate potential consequences of casualties, direct economic loss and downtime caused by building damage, including both structural and nonstructural components and systems. Other tools provide loss information that can be used to help establish acceptable nonstructural performance criteria and guide design decisions.

This chapter presents a brief summary of several practical guidelines for structural engineers; mechanical, electrical, and plumbing (MEP) engineers; architects; and contractors. Performance-based design tools currently in use are also presented.

5.1 Practical Guidelines

Practical guidelines are intended to provide easy-to-understand information that further explains existing building code provisions or deem-to-comply rules that comply with the building codes by providing prescriptive solutions to mitigate:

- direct damage to nonstructural components;
- secondary damage caused by the failure of nonstructural components; and
- potential life safety issues created by the failure of nonstructural components.

5.1.1 *FEMA E-74, Reducing the Risks of Nonstructural Earthquake Damage – A Practical Guide*

The primary purpose of FEMA E-74, *Reducing the Risks of Nonstructural Earthquake Damage – A Practical Guide* (FEMA, 2012d), is to explain the sources of nonstructural earthquake damage and to describe methods for reducing the potential risks in simple terms. The guide is aimed at a wide audience with varying needs including design professionals, building owners, facility managers, maintenance personnel, store or office managers, risk managers, and safety personnel. It addresses nonstructural issues typically found in schools, office buildings, retail stores, hotels, data centers, hospitals, museums, and light manufacturing facilities. The current edition of FEMA E-74 was published online in 2012 and is the fourth in a series, which dates back to the 1980s.

FEMA E-74 aggregates information on nonstructural components and systems from a wide range of sources and summarizes it in a format usable by individuals on a system-wide or component basis. Chapter 6 of FEMA E-74 contains a summary of over fifty nonstructural components. For each component, building code and retrofit provisions are summarized, a collection of earthquake damage photographs is provided and sample details for reducing seismic damage are offered. The document also contains: (1) an extensive list of references; (2) performance specifications that can be adapted to project needs; and (3) responsibility matrices to facilitate comprehensive planning of nonstructural components and systems in construction projects.

In April 2015, a FEMA-sponsored webinar on FEMA E-74, attracted over 900 participants from more than 30 countries and validates the widespread use of FEMA E-74. The effectiveness of FEMA E-74 as a tool in reducing nonstructural damage is attributed to the easy access it provides to useful up-to-date information in a simple, easy-to-understand format.

The continued effectiveness of FEMA E-74 as resource document for nonstructural loss reduction will be in part related to continual updating to keep it relevant. Integration of code changes, new observations of nonstructural damage and improved details will allow FEMA E-74 to continue to contribute to loss reduction.

There appears to be an opportunity to expand usage of FEMA E-74 to a broader group of design professionals including architects, mechanical, electrical, plumbing, fire protection and others. Focused outreach efforts may be needed to penetrate these constituent groups.

5.1.2 FEMA 412, Installing Seismic Restraints for Mechanical Equipment; FEMA 413, Installing Seismic Restraints for Electrical Equipment; and FEMA 414, Installing Seismic Restraints for Duct and Pipe

This series of guidebooks, FEMA 412, *Installing Seismic Restraints for Mechanical Equipment* (FEMA, 2002); FEMA 413, *Installing Seismic Restraints for Electrical Equipment* (FEMA, 2004c); and FEMA 414, *Installing Seismic Restraints for Duct and Pipe* (FEMA, 2004d) show installers how to attach mechanical, electrical, and plumbing (MEP) components in a building to minimize potential earthquake damage. Many illustrations are provided to provide hands-on guidance relevant to the trades ultimately responsible for installation. The guidelines help installers understand the importance of the details used for common types of equipment and contain warnings to prevent improper installations.

5.1.3 ASHRAE RP-812, Practical Guide to Seismic Restraint

The American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE) publishes RP-812, *Practical Guide to Seismic Restraint* by (Tauby and Lloyd, 2012), which shows installers how to attach mechanical, electrical, and plumbing components in a building to minimize potential earthquake damage. The guide also explains how to design, specify, and install seismic restraints for mechanical and plumbing systems in buildings to meet the requirements of ASCE/SEI 7-10, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2010).

In addition to an introduction to the fundamentals of earthquakes and seismic restraints, the guide includes information on shake table testing to satisfy special certification requirements for Designated Seismic Systems, guidance on using flexible connectors to help mitigate failure of equipment piping connections, and illustrated examples and calculations for design of restraints for suspended and floor-mounted equipment, ducts, piping, cooling towers, and similar components.

5.1.4 Safer Schools: Guide and Checklist for Nonstructural Earthquake Hazards in California Schools

The *Safer Schools: Guide and Checklist for Nonstructural Earthquake Hazards in California Schools* (CalOES, 2003) was published to reduce seismic hazards associated with the nonstructural components of school buildings, including mechanical systems, ceiling systems, partitions, light fixtures, furnishings, and other building contents. This document was subsequently revised in 2011 by the California Emergency Management Agency (CalEMA, 2011). The guidelines are intended to be used primarily by installers, facility managers and school staff to reduce the potential for fatalities and injuries caused by nonstructural components and systems.

Illustrations provide guidance for proper installations of components commonly found in and around classrooms. Seismic restraint solutions are generally easy to implement. The document also contains “quick action items” that can be easily adopted to immediately improve classroom safety.

An Earthquake Hazards Checklist is also provided, which can be used to conduct nonstructural surveys and identify components or systems requiring improvement to protect safety. Based on observations of nonstructural damage to schools in the 2014 South Napa Earthquake and prior earthquakes, annual completion of the checklist on a room-by-room basis for each school in a seismically active location would appear to help reduce future losses and protect students from potentially life-threatening damage.

5.2 Performance-Based Design Tools

Performance-based design tools provide design professionals with information needed to design nonstructural components to meet specified performance criteria that could be above and beyond what is required in the building code, but meets the needs of the building owner.

5.2.1 FEMA P-58, Seismic Performance Assessment of Buildings

The FEMA P-58 project developed a methodology for seismic performance assessment of buildings over a 10-year study funded by FEMA and performed by the Applied Technology Council. The resulting products were organized in a three-volume set, collectively referred to as FEMA P-58, *Seismic Performance Assessment of Buildings, Methodology and Implementation* (FEMA, 2012a; 2012b; 2012c). The methodology is tailored to building-specific analysis, and is based on development of a building performance model, which is an organized collection of data used to define the building assets at risk and their exposure to seismic hazards. This includes structural components and assemblies as well as nonstructural components, systems and contents. Components that are vulnerable to damage are assigned a fragility specification that includes information on damage states and related consequences in terms of repair costs, repair time and fatalities and injuries. The database of fragilities was developed from test data, where available, and expert opinion. This database contains groups of nonstructural components and systems reflecting the range of damageable items generally found in buildings, and installation conditions common to buildings constructed within the past 50 years.

Site specific earthquake hazards are defined in different ways depending on the type of assessment desired. Practical implementation of the methodology can be accomplished using an electronic Performance Assessment Calculation Tool (PACT) for performing the probabilistic computations and accumulation of losses or commercially available software.

The results of a FEMA P-58 assessment provide an understanding of the vulnerability of nonstructural components and systems in terms of repairs costs, repair time, and risk of injury or fatality. Availability of this information can serve as an opportunity to guide design decisions that will help meet desired performance objectives. For example, if unacceptable losses are calculated, additional analyses can be performed to examine the impact of different nonstructural details or structural characteristics. A sample FEMA P-58 output illustrating expected annual losses and relative contributions to loss are shown in Figure 5-1.

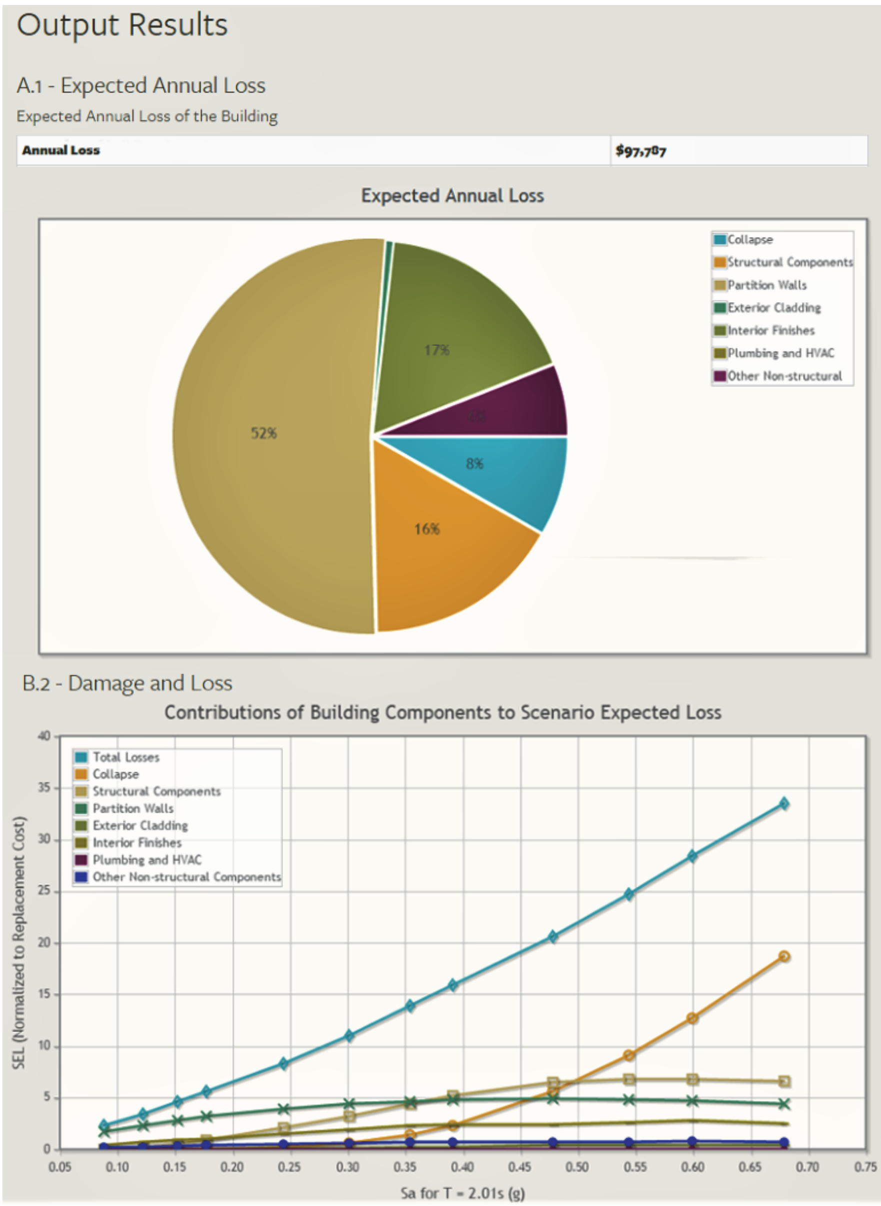


Figure 5-1 Sample presentation of FEMA P-58 assessment data, illustrating expected annual losses and relative contributions to loss.

Chapter 6

Simulated Seismic Testing of Nonstructural Components and Systems

To complement analytical studies and field observations, experimental studies of nonstructural components and systems (NCSs) have been recently conducted to advance the state of understanding with regards to their seismic performance. Depending on the experimental scope and objective, the simulated seismic testing of NCSs are categorized as: (1) system-level building shake table tests; (2) component tests; and (3) designated seismic system qualification tests. This chapter summarizes the seismic tests of NCSs in the United States and Japan in the last decade. In addition, this chapter also describes a research effort for developing a seismic response and performance database of acceleration-sensitive NCSs.

6.1 System-Level Building Shake Table Tests

This section summarizes three system-level shake table experimental research projects on nonstructural components and systems in the recent decade: (1) full-scale building test at the University of California San Diego (UCSD); (2) full-scale building test at the National Research Institute for Earth Science and Disaster Prevention (NIED) E-Defense Shake Table Facility in Miki City, Japan; and (3) full-scale testbed for ceiling-piping-partition walls at the University of Nevada Reno. While the UCSD tests incorporated a variety of acceleration-sensitive and displacement-sensitive nonstructural systems in the test scope, the other two tests primarily focused on several specific systems (e.g., partition walls, suspended ceilings, fire sprinkler systems, and building contents). These three projects funded, in part, by the National Science Foundation provide well-documented experimental datasets that are useful for researchers and engineers in future activities.

6.1.1 University of California San Diego Full-Scale Five-Story Building Tests

A unique research collaboration between academia, government, and industry, coined the Building Nonstructural Components and Systems (BNCS) project, was undertaken to contribute to understanding the earthquake resiliency of nonstructural components and systems (Hutchinson et al., 2013; BNCS, 2014). The centerpiece of this research effort involved shake table testing of a full-scale five-story reinforced concrete building outfitted with a large variety of essential NCSs. These tests, which were completed in 2012, contribute a wealth of high-resolution physical data to the earthquake and fire engineering communities and will provide direct input to modeling tools, future design codes, and construction practices.

This landmark experimental program was performed using the Large High-Performance Outdoor Shake Table (LHPOST) (Van den Einde et al., 2004) at the UCSD. The shake table test program involved two test phases: (1) earthquake shaking while the building was isolated at its base from the shake table (BI Phase); and (2) earthquake shaking while the building was fixed at its base to the shake table (FB Phase). The test building was outfitted with a variety of essential NCSs, which included operable egress systems (elevator and steel stairs), a complete façade, a broad array of architectural layouts, and two floors of the building equipped as medical facilities. Additional details of the experimental program, seismic testing phases, and findings may be found in five technical reports (Hutchinson et al., 2013; Chen et al., 2013a; Chen et al., 2013b; Pantoli et al., 2013a; Pantoli et al., 2013b).

Test Building

The test building, shown in Figure 6-1a, is a cast-in-place five-story reinforced concrete structure with moment resisting frames providing lateral resistance in the direction of shaking. Ground motions developed for a site in Southern California with a maximum considered earthquake (MCE) ground motion spectrum for a Site Class D (stiff) soil condition, a short-period spectral acceleration S_{MS} = 2.1g and a one-second spectral acceleration S_{M1} = 1.4g were used for the building design. Performance targets of a 2.5% peak interstory drift ratio and a maximum peak floor acceleration between 0.7 and 0.8g were selected during the conceptual design phase.

The building configuration was two bays in the longitudinal direction and one bay in the transverse direction, with a plan dimension of 36 feet (11.0 m) by 21.5 feet (6.6 m). As shown in Figure 6-1c, the lateral system is two moment resisting frames in the east bays to resist longitudinal shaking direction along with two shear walls within the interior of the building to resist transverse lateral and partial torsional loads. The floor diaphragm provided two major openings: one on the northwest to facilitate a full-height elevator shaft (from the first to the fifth floors) and the other on the southeast to accommodate stairs (from the second floor to the roof). The building provided two useful spaces on each floor to accommodate a broad variety of NCSs. Occupancies designated for these spaces are briefly discussed later. As shown in Figure 6-1d, the floor-to-floor height was 14 feet (4.3 m) at each level, resulting in a building height of 70 feet (21.3 m) above the foundation. The estimated weight (excluding the foundation) was 680 kips (3,009 kN) for the structural skeleton and 1,010 kips (4,492 kN) including all nonstructural components and systems, and the estimated weight of the foundation was 420 kips (1,868 kN).

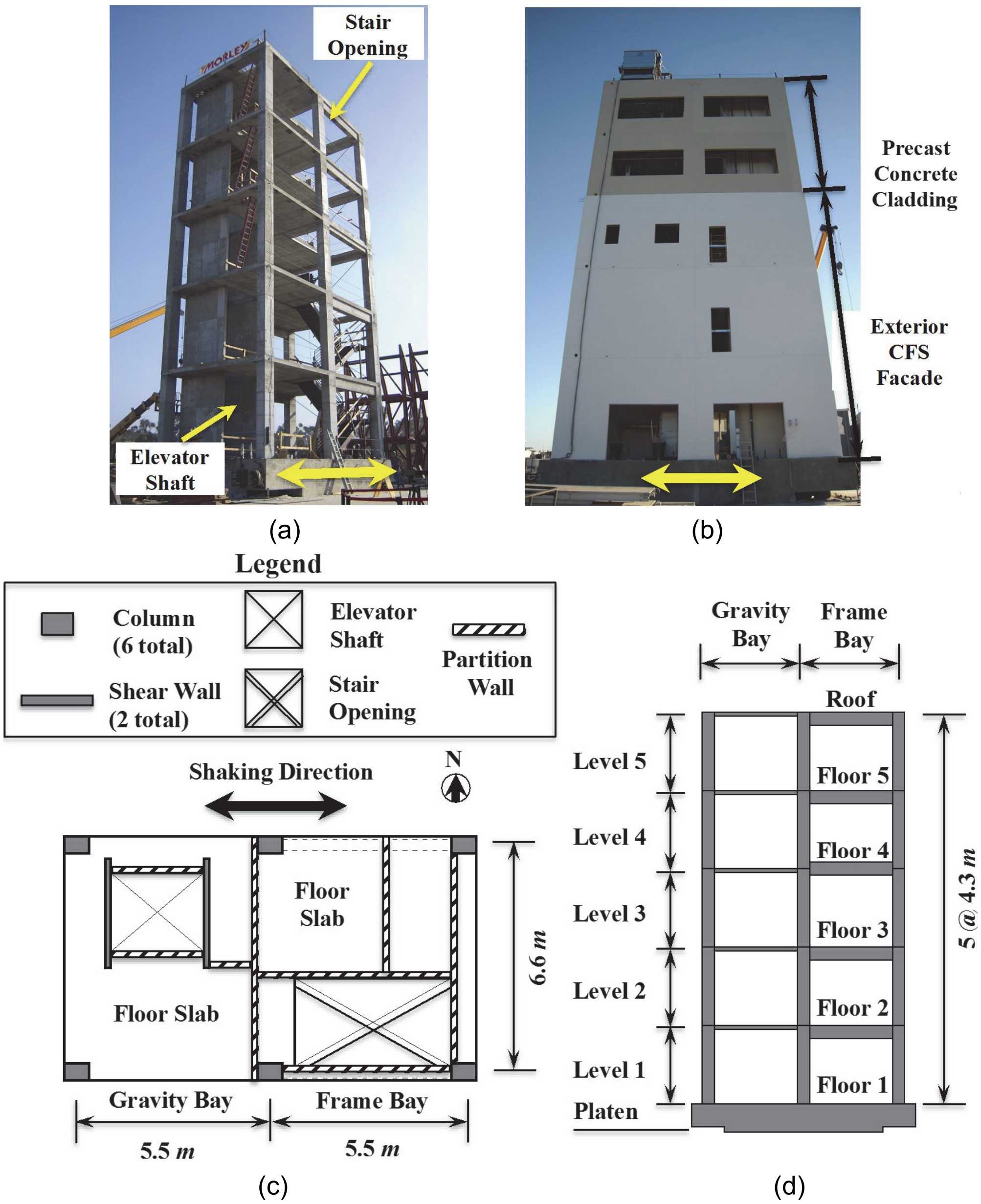


Figure 6-1 Test building: (a) structural skeleton of the building; (b) test building at completion; (c) typical floor plan layout (note that interior partition wall layout varies from floor to floor); and (d) elevation (Wang et al., 2015).

Nonstructural Components and Systems

The test building was outfitted with a broad variety of NCSs following completion of the construction of its structural skeleton. The major NCSs installed on the test building included:

- Two full-height egress systems: (1) a prefabricated steel stair system on the northwest side; and (2) a functioning passenger elevator on the southeast. The stairs provided access to all floors including the roof, whereas the elevator provided access to all floors except the roof as shown in Figure 6-1a.

- Two different types of architectural façades, shown in Figure 6-1b, were installed on the building: (1) balloon framed cold-formed steel (CFS) studs overlaid with a synthetic stucco at the lower three levels; and (2) precast architectural concrete cladding at the upper two levels.
- Interior architectural components based upon occupancy of the various spaces: (1) level one was designated as a utility floor allowing sufficient space for placement of electrical services and installation of four large access doors (two steel rolling doors and two sectional garage doors) placed on the longitudinal faces of the building; (2) level two was detailed as both a laboratory and residential space; (3) level three was planned for live fire tests and therefore provided with the most complete detailing of partition walls, ceilings, plenum space and associated finish work; and (4) levels four and five were outfitted as hospital floors.
- Fire protection and gas piping; heating, ventilation, and air-conditioning ducts; and electrical service systems common to a building were installed to varying degrees of completeness at each floor of the building.
- Various types of equipment were installed on each level of the test building based on the suggested occupancy of each floor level, while the roof supported a cooling tower, penthouse, and air-handling unit.

Nonstructural Design Criteria

Pretest numerical simulations of the test building were conducted to estimate design forces and deformations imposed during the design event earthquake (FB-5:DEN67 as later used in the test phase). Although this study only used one design event motion to develop the nonstructural design criteria, the actual development of a building-specific design criteria is better suited within a probabilistic framework which would require multiple motions to quantity the uncertainty of building response. This pretest simulation effort included developing two nonlinear finite element models independently: (1) a macro-element based model implemented in *OpenSees* (software developed and distributed by the Pacific Earthquake Engineering Research Center; and (2) a detailed finite element model in DIANA (Software developed and sold commercially by TNO DIANA). Using these test results, building-specific design recommendations were back calculated for product suppliers to support the design of their nonstructural systems specifically to the target design motion. Specific recommendations of acceleration-sensitive and displacement-sensitive nonstructural components are described in the following paragraphs, and additional details regarding the pretest simulation and nonstructural design criteria may be found in Wang et al. (2013).

Recommended Design Interstory Drift Ratios

Unlike ASCE/SEI 7-10 (ASCE, 2010) provisions that use a constant value (which is normally 2.0 to 2.5% dependent on the type of structure) as the maximum drift level for all floors, the recommended design interstory drift ratios for drift-sensitive components considers the variation of the interstory drift responses at different levels. These recommended values were determined by taking the arithmetic average of the peak interstory drift results as estimated with the two numerical models. As shown in Table 6-1, the achieved drift ratios agree well with the recommended drifts at the lower levels, but were roughly one-half of the recommended values for the upper two levels and therefore not as demanding.

Table 6-1 Recommended Design and Achieved Interstory Drift Ratio Percentages

Floor Level	1	2	3	4	5
OpenSees	2.3	2.9	2.6	2.0	1.0
DIANA	1.2	2.2	2.4	2.0	1.1
Recommended	1.8	2.6	2.5	2.0	1.1
Achieved	2.6	2.8	2.1	1.1	0.5

Recommended Design Floor Spectral Accelerations

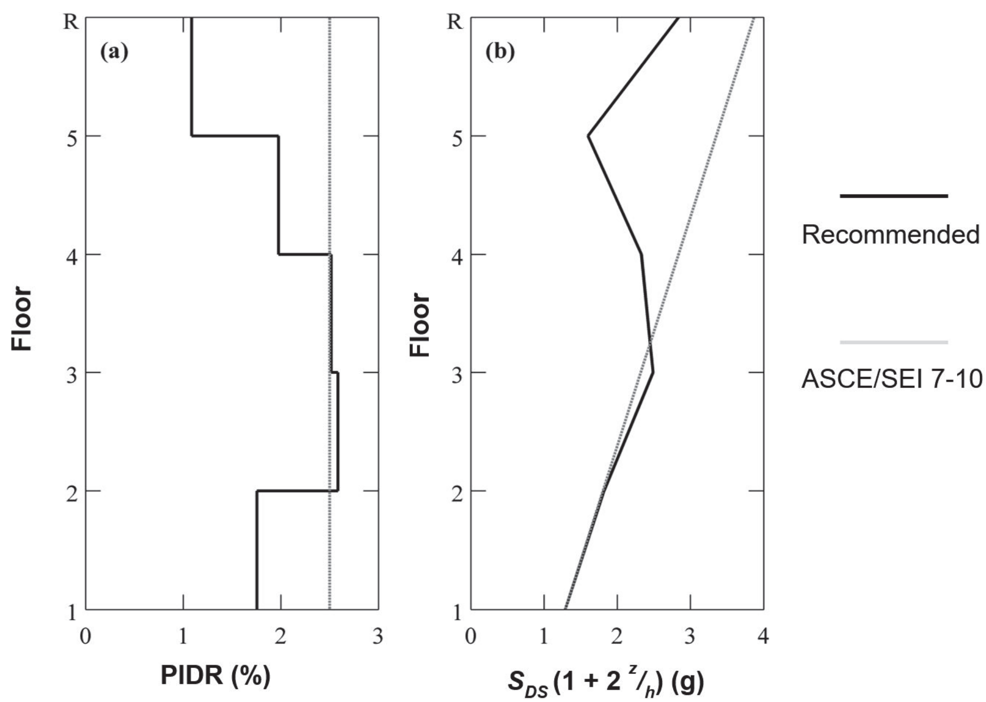
The recommended design floor spectral accelerations, on the other hand, were calculated based on the floor acceleration spectra of the two models. To determine these design values, the 5%-damped floor acceleration spectra are calculated first using the acceleration time histories from each model. These spectral values from the two models within the period range from 0.06 to 0.6 seconds were subsequently averaged at an increment of 0.02 seconds. Note that 0.06 seconds was suggested as the period defining a rigid nonstructural component in the design provisions (e.g., ASCE/SEI 7-10). The mean and standard deviation of the averaged spectral values within the specified period range were then statistically determined. Finally, the recommended design floor spectral accelerations were determined as the mean plus one standard deviation over the selected period range as shown in Table 6-2. These recommended values were utilized to replace the term S_{DS} $(1 + 2\,{}^{z}\!/_{h})$ in ASCE/SEI 7-10 Eq. 13.3-1 and facilitate estimation of the design force demands of the nonstructural components. Table 6-2 also provides the achieved "short-period" spectral accelerations at each floor, which are determined as the mean plus one standard deviation of the achieved floor spectral accelerations over the period range between 0.06 and 0.6 seconds.

A comparison between the recommended values and those conforming to ASCE/SEI 7-10 is shown in Figure 6-2. It is assumed that the prescribed interstory drift is 2.5% for all levels. The spectral acceleration, S_{DS}, is taken as 1.29g at the base (mean plus

Table 6-2 Recommended Design and Achieved Floor Spectral Accelerations (in g's)

Level	1	2	3	4	5	Roof
Mean	1.16	1.52	1.89	1.69	1.37	2.25
Standard Deviation	0.13	0.31	0.60	0.65	0.23	0.59
Recommended	1.29	1.82	2.49	2.33	1.60	2.84
Achieved	1.58	1.81	2.05	1.39	1.17	1.97

one standard deviation of the spectral accelerations of the design event earthquake motion over the period range between 0.06 and 0.6 seconds). Rather than employing a constant interstory drift demand at each floor, the recommended drift ratios consider the variation of the interstory drift responses at different levels that reasonably reduces the seismic drift demands at the higher levels. Likewise, the shape of the recommended spectral accelerations deviates from the linear distribution at the higher levels and thus lowers the force demands to the components at the higher levels.



(a) (b)

Figure 6-2 Comparison between the recommended design criteria and those conforming to ASCE/SEI 7-10: (a) interstory drift ratio; and (b) $S_{DS}(1 + 2\ ^{z}/_{h})$ (Wang et al., 2013).

Test Protocol

The test building was subjected to a sequence of dynamic input motions at the base of the building, while the building was first tested in base isolated (BI) and subsequently in fixed base (FB) configurations. These excitations, 13 earthquake motion tests, 31 low amplitude white noise base excitation tests and 45 pulse-like base excitation tests, were conducted using the UCSD LHPOST. Each of the input motions was applied in the east-west direction using the single-axis shake table, whose axis coincided with the longitudinal axis of the building. Shown in the chronology of their application, Table 6-3 summarizes the earthquake test motions during the two test phases. With the exception of motions BI-4:SP100 and BI-7:ICA140, all motions in the BI test phase were also imposed in the FB test phase.

Table 6-3 Summary of Earthquake Test Motions

2012 Date	Earthquake Event – Site – Scaling (%)	Base Condition	Name
April 16	1994 Northridge – Canoga Park – 100%	Isolated	BI-1:CNP100
April 16	1994 Northridge – LA City Terrace – 100%	Isolated	BI-2:LAC100
April 17	1994 Northridge – LA City Terrace – 100%	Isolated	BI-3:LAC100
April 17	2010 Maule (Chile) – San Pedro – 100%	Isolated	BI-4:SP100
April 26	2007 Pisco (Peru) – Ica – 50%	Isolated	BI-5: ICA50
April 27	2007 Pisco (Peru) – Ica – 100%	Isolated	BI-6: ICA100
April 27	2007 Pisco (Peru) – Ica – 140%	Isolated	BI-7:ICA140
May 7	1994 Northridge – Canoga Park – 100%	Fixed	FB-1:CNP100
May 9	1994 Northridge – LA City Terrace – 100%	Fixed	FB-2:LAC100
May 9	2007 Pisco (Peru) – Ica – 50%	Fixed	FB-3:ICA50
May 11	2007 Pisco (Peru) – Ica – 100%	Fixed	FB-4:ICA100
May 15	2002 Denali earthquake – TAPS Pump Station #9 – 67%	Fixed	FB-5:DEN67
May 15	2002 Denali earthquake – TAPS Pump Station #9 – 100%	Fixed	FB-6:DEN100

The earthquake input motions selected in the test program encompassed a broad range of characteristics including different frequency contents as well as varied strong motion durations and amplitudes. Recorded motions consisted of those from the subduction zone of South America, the coast of California, and the central area of Alaska, representing the ground motion characteristics with varied seismicity. The motions recorded from the earthquakes in Chile and Peru were actual recordings that were scaled only in amplitude, while the Denali test motion was scaled in both frequency and amplitude (scaled using spectral matching) to a targeted response spectrum with Site Class D soil conditions for the selected site. A 100%-scale factor for the Denali motion implies that the response spectra of the test motion matches a maximum target response spectrum with spectral acceleration values of S_{MS} = 2.1g and S_{M1} = 1.4g, whereas 67%-scale factor for the motion indicates that the test motion has a response spectrum that was intended to match the design event response spectrum and impose the performance targets set for the building. All the Northridge earthquake input motions were spectrally matched to achieve a seismic hazard level with a return period of 43 years (approximately 20% of the maximum target response spectrum). The seismic motions were designed and applied to the building and its NCSs with the intent to progressively increase the seismic demands on the building and NCSs in both the BI and FB test phases, while minimizing the impact of the lower intensity motions on the failure response mechanisms developed in the specimen under the highest intensity motion. Details of the input earthquake motions in these test phases are discussed in Chen et al. (2013a).

Test Results

This section first presents the test results in the following two parts: (1) a summary of the response of the building as well as the seismic performance of the building from an overall perspective; and (2) a case study of the roof-mounted cooling tower, to illustrate how results from this full-scale test program can be utilized to understand the performance of a specific NCS.

Test Building Response and Overall Nonstructural Performance

The measured peak floor accelerations (PFAs) and peak interstory drift ratios (PIDRs) of the test building under the two test configurations is shown in Figure 6-3. It is noted that the floor response as presented in the figure is taken as the averaged response of the four corners of the building. Since the seismic demands on the building were relatively low with peak interstory drift ratio (PIDR) of less than 0.4% and peak floor acceleration (PFA) of less 0.3g in the BI test phase, the building sustained only minor damage to its most brittle nonstructural components such as partition walls and very little damage to its structural components. In the FB test phase, the earthquake motions were applied with increasing intensity to progressively damage the structure. Test FB-5:DEN67 is considered as design event for the test building since the design target PIDR of about 2.5% was achieved during this test, while test FB-6:DEN100 represents a well above design event scenario as the achieved PIDR was as much as 6%. The building sustained extensive damage during the last two FB tests as a result of the large seismic drift demands. Physical damage during test FB-6:DEN100 included fracture of the longitudinal reinforcement at the ends of the frame beams and punching shear mechanisms at the slab-column interfaces of the second and third floors. This resulted in the development of a soft-story mechanism at the lower levels of the test building (also known as an intermediate failure mechanism).

The observed damage of the NCSs was classified as three discrete damage states: minor, moderate or severe, and are subsequently correlated with either measured peak interstory drift ratio (PIDR) or peak floor acceleration (PFA). Several NCSs in the test program demonstrated quite good performance, attaining design expectations and remaining functional despite the very large demands imposed upon them. These

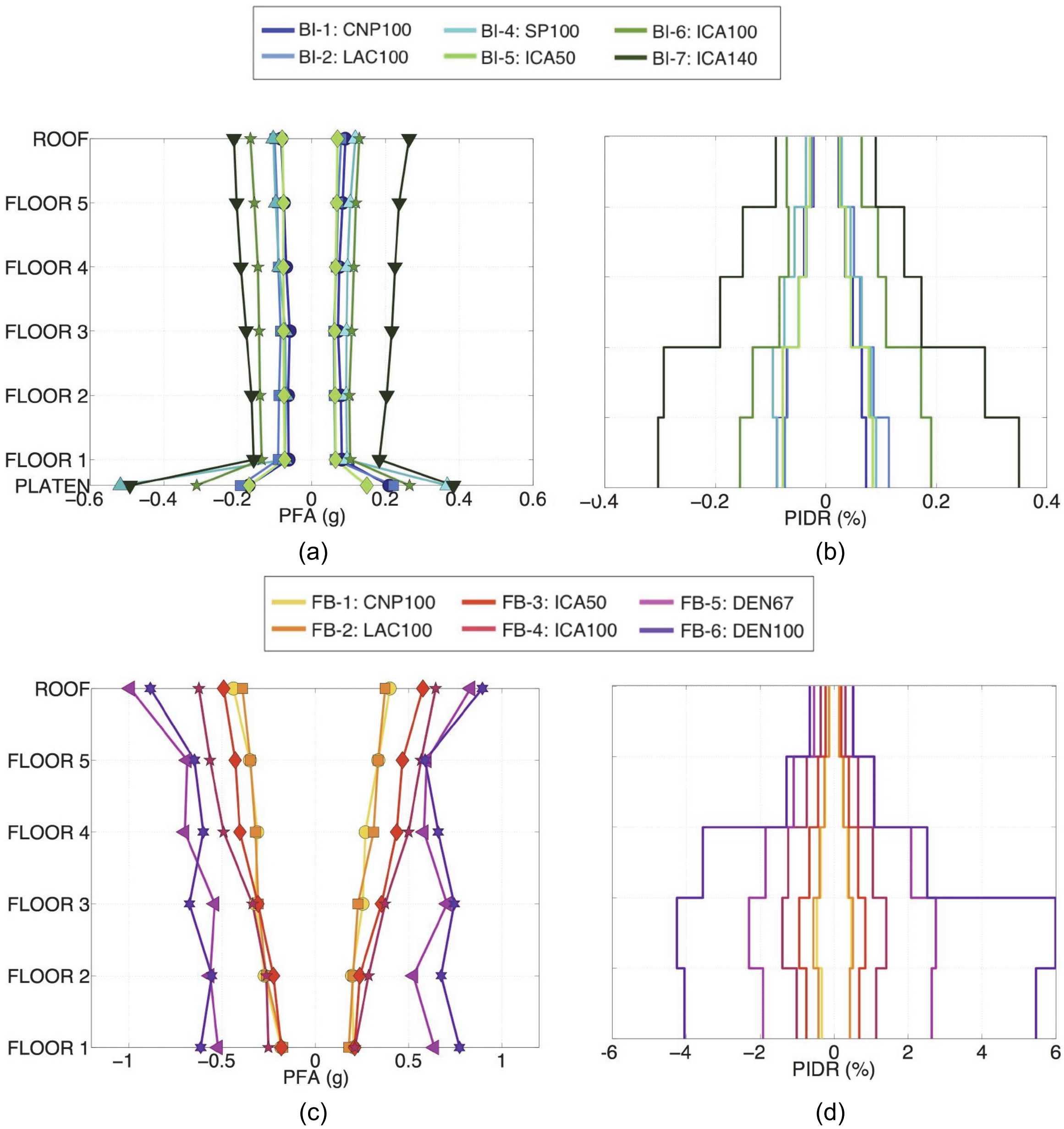


Figure 6-3 Measured building response in the longitudinal direction: (a) peak floor accelerations during the base isolated tests; (b) peak interstory drift ratios during the base isolated tests; (c) peak floor accelerations during the fixed base tests; and (d) peak interstory drift ratios during the fixed base tests (Chen et al., 2015).

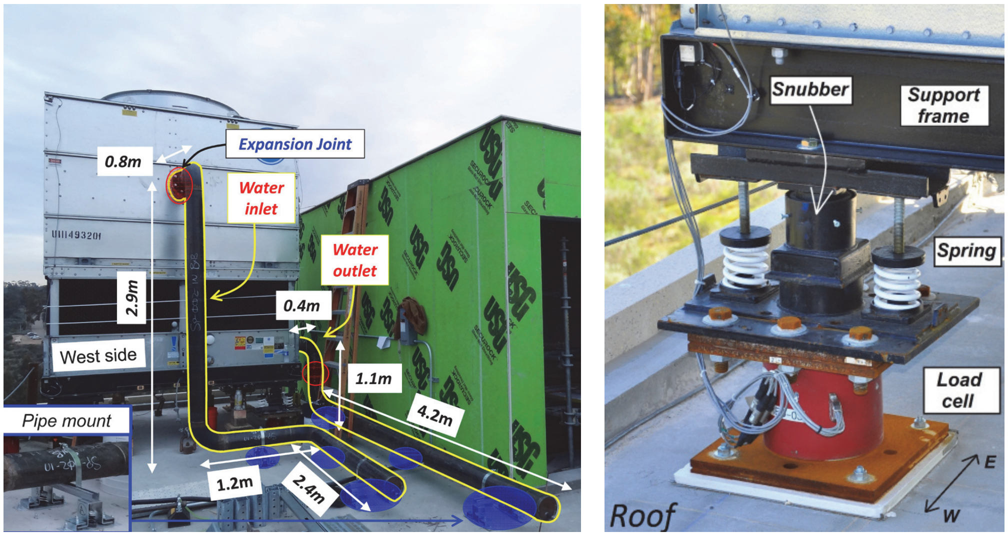
NCSs, which were limited to mostly minor damage throughout the test program, included the fire sprinkler system, seismically designed ceilings, roof mounted equipment, and restrained contents. In addition, the precast concrete cladding panels and the passenger elevator were subjected to moderate or severe damage only during the last two earthquake motions in the FB test phase. In contrast, select NCSs installed in the test building attained unacceptable levels of damage. These components were:

- **Stairs.** Prefabricated steel stairs, which were arranged in a scissor assembly and designed to accommodate interstory drift via a structural steel angle with a reduced area suffered damage. This performance renders concern for the stairway operability at PIDRs as low as 0.8%, which is well below design interstory drifts of the test building. Such severe damage manifested in the form of separation between the flights and supporting building slabs and in all cases was due to weld fracture.
- **Cold-formed steel (CFS) balloon framing overlaid with synthetic stucco.** Three levels of the building were clad in a CFS stud "balloon framed" stucco-finished façade, the major components of which included its gypsum boards, CFS attachment clips, and exterior insulation and finishing system (EIFS). Damage manifested in the form of fracture of gypsum boards, tearing of corners of the stucco-finish, and detachment of a large number of connection clips.
- **Medical equipment.** Two levels of the building were designed to mimic the layouts of a surgery suite and intensive care unit, with equipment staged as either restrained or unrestrained and placed in a variety of common orientations. Dangerous toppling of several unrestrained pieces of equipment was observed at PFA as low as 0.56g, amplitudes below the design floor acceleration.

Specific Nonstructural Performance Case Study: Cooling Tower

A cooling tower system was installed at the roof of the test building and thus tested along with the building during both the BI and FB test phase (Astroza et al., 2015). As shown in Figure 6-4a, the cooling tower consisted of a heat transfer module, a water collection module, an air handling module, a fan motor, and a fan. The dimensions of the cooling tower were 9 feet (2.73 m) by 7 feet (2.14 m) in plan and 10.5 feet (3.25 m) height. The tower was mounted on a steel base frame and four isolation/restraint (I/R) supports as shown in Figure 6-4b. The total weight of the cooling tower for empty and operational (filled with water) conditions was 3.5 kips (15.6 kN) and 6.3 kips (27.9 kN), respectively. During the seismic tests, the cooling tower was instrumented with two tri-axial accelerometers at the northwest and southeast corners at the top of the cooling tower, and one tri-axial accelerometer was installed on the roof slab underneath the cooling tower. In addition, load cells, as shown in Figure 6-4b, were installed at the four supports of the tower to measure the moment and shear forces in two horizontal directions and the axial force.

During the construction phase, a series of free vibration tests and ambient vibration tests were conducted on the cooling tower, considering both its empty and operational (water filled to its full capacity) conditions. The purpose of these tests was to understand the modal parameters (e.g., natural frequencies, damping ratios) of the cooling tower using system identification techniques and presented in Table 6-4. The first three identified modes primarily represent translational rigid body modes of



(a) (b)

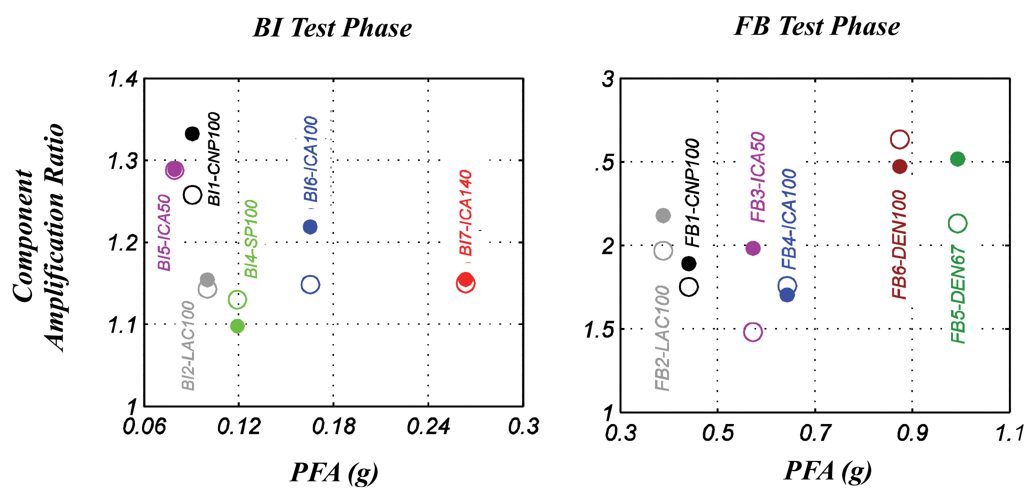
Figure 6-4 Cooling tower test assembly: (a) photograph with annotation; and (b) the isolation/restraint supports (Astroza et al., 2015).

the cooling tower above the supports (with slight rocking effects), in the vertical, transverse, and longitudinal directions, respectively. The fourth mode represents a torsional mode, while the fifth and sixth correspond to the rocking modes. As a result of the mass contribution of water, the natural frequencies of the cooling tower when fully filled with water were slightly lower than their counterparts for the tower in its empty condition. The damping ratios range between 2.0 and 5.0%, which remained almost identical when the tower was filled with water.

Table 6-4 Identified Natural Periods, Frequencies, and Equivalent Damping Ratios of the Cooling Tower System (Astroza et al., 2015)

		Empty Tower			Water-Filled Tower		
Mode	Direction	*Period (second s)*	*Frequency (Hz)*	*Damping Ratio (%)*	*Period (seconds)*	*Frequency (Hz)*	*Damping Ratio (%)*
1	Vertical (UD)	0.21	4.87	3.5 - 4.5	0.22	4.65	3.5 - 4.5
2	Transverse (EW)	0.18	5.64	3.0 - 4.0	0.19	5.26	3.0 - 4.0
3	Longitudinal (NS)	0.15	6.70	2.5 - 3.5	0.15	6.52	2.5 - 3.5
4	Torsion	0.12	8.48	4.0 - 5.0	0.12	8.28	4.5 - 5.5
5	Rocking Around Transverse Axis	0.10	10.31	4.0 - 5.0	0.10	9.84	4.0 - 5.0
6	Rocking Around Longitudinal Axis	0.07	14.03	2.0 - 3.0	0.07	14.02	2.0 - 3.0

The acceleration measurements of the building and the cooling tower during the seismic test phase allowed for studying the dynamic amplification effects of the cooling. The component amplification ratios of the cooling tower in the shaking direction during the BI and FB test phases is shown in Figure 6-5. The component amplification ratio is defined as the ratio of the peak accelerations recorded at the top of the cooling tower and the corresponding peak input acceleration recorded at the roof of the building (solid and hollow circles represent the two measurement points at the top of the cooling tower). During the BI test phase, the cooling tower experienced low amplification effects, since the component amplification ratio ranged between 1.1 and 1.3 as shown in Figure 6-5a. This is likely due to the fact that the response of the structure was concentrated in a frequency range that was much lower than the natural frequencies of the cooling tower. However, the component amplification ratio of the cooling tower increased significantly and ranged between 1.5 and 2.6 as shown in Figure 6-5b. The largest acceleration recorded at the top of the tower (approximately 2.6g) occurred during test FB5:DEN67 (design event earthquake). It is noted that the period of the second longitudinal mode of the building at the beginning of the FB test phase ($T = 0.17$ seconds) was close to the translational periods of the cooling tower ($T = 0.19$ seconds and $T = 0.15$ seconds), thus leading to much more significant amplification effect for the cooling tower compared with those in the BI test phase.



(a) (b)

Figure 6-5 Component amplification ratios of the cooling tower system in: (a) base-isolated test phase; and (b) fixed-base test phase (Astroza et al., 2015).

In addition to measured acceleration results of the cooling tower system, the experimental results included the measured displacement at the base of the cooling tower as well as the force measurement of the supports. Visual inspection and functionality checks were also conducted following each earthquake test. Although a maximum of 30% water loss was detected during the tests, the cooling tower system experienced no visible damage or loss of functionality during all tests. Additional discussions of the test results and the seismic performance and of the cooling tower can be found in Astroza et al. (2015).

Test Dataset

The test building and NCSs were monitored with digital still cameras, more than 80 video cameras, 500 analog sensors, and a global positioning system (GPS). Data collected from these monitoring systems are archived and publicly available within the online NEES database:

- Project "NEESR-CR: Full-Scale Structural and Nonstructural Building System Performance during Earthquakes": containing photographs, videos and data form analog sensors (Hutchinson et al., 2014); and
- Project "Low-Cost, Strong-Motion Sensors packages to Obtain Full Spectrum Waveforms for Earthquake Early Warning and Structural Monitoring Applications" containing GPS data [DOI: 10.4231/D3V97ZR5H].

6.1.2 E-Defense Full-Scale Five-Story Building Tests

A collaborative research project, between the Network for Earthquake Engineering Simulation (NEES) and the National Research Institute for Earth Science and Disaster Prevention (NIED) of Japan, was conducted at the E-Defense Shake Table Test Facility in 2011 (Guzman and Ryan, 2015a). This project consisted of a system-level full-scale shake table experiments of a five-story steel moment frame building. The building was tested in three different configurations: (1) base isolated with triple pendulum bearings (TP configuration); (2) base isolated with a hybrid isolation system (hybrid configuration); and (3) fixed-base configuration. The primary objective of these shake table experiments was to demonstrate the effectiveness of base isolation to protect the building structure as well as the nonstructural components in high-intensity earthquakes. Although the lateral load system of the building was designed and detailed according to Japanese code provisions, the nonstructural systems were designed and installed in compliance with United States practice.

Test Building

The test building was a full-scale five-story steel moment frame building and is shown in Figure 6-6a. The building consisted of two bays in each horizontal direction, with a plan dimension of 39 feet (12 m) by 33 feet (10 m) as shown in Figure 6-6b and a total height of approximately 52 feet (16 m) as shown in Figure 6-6c. The bay widths were asymmetric in the longitudinal direction (23 feet (7 m) and 16 feet (5 m) spans) to promote torsion of the structure. All primary beam-column connections were fully welded and restrained moment connections. The floor system consisted of reinforced concrete slabs cast on corrugated metal decking in floors two to five and cast on a flat steel deck on the roof. The concrete slabs were connected to primary beams by shear studs. Steel plates weighing 120 kips (535 kN) were placed on the roof in an irregular configuration to enhance the asymmetry of the building specimen. The weight of the superstructure was about 1,200 kips (5,300 kN) including the additional roof mass and the participating mass at the base level. Design details of the test building are discussed further in Ryan et al. (2013).

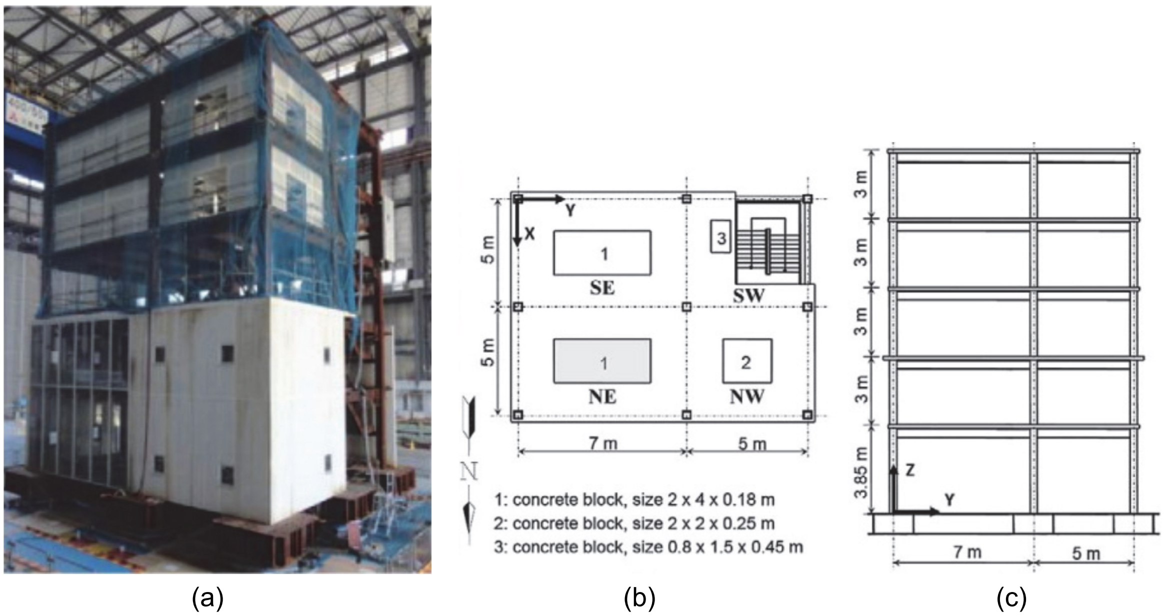
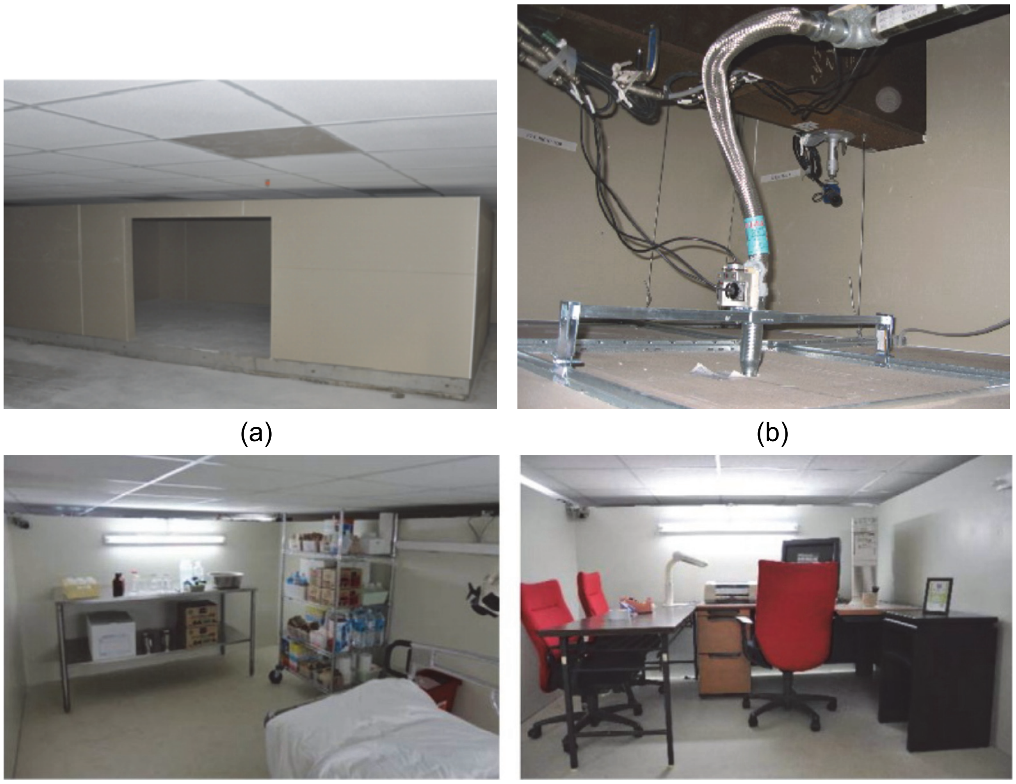


Figure 6-6 Five-story steel moment frame specimen: (a) photograph; (b) typical plan view from floors two to five; and (c) elevation view (Guzman and Ryan, 2015a).

Nonstructural Components and Systems

Nonstructural components incorporated within the test building encompassed an integrated of suspended ceilings, partition walls, and sprinkler piping subassemblies on each of the upper two levels of the building (Soroushian et al., 2016) as well as a full-story precast concrete cladding 3D panel and flat panel that together represented a corner subassembly (McMullin et al., 2012). The sprinkler piping subassemblies installed in the upper two stories were nearly identical in configuration on each story. One key difference is that the ceilings attached below the fifth floor were unbraced, while the ceilings attached below the roof were braced using compression posts and diagonal splay wires. The subassembly at each story included approximately 300 feet (90 m) of partition walls, 900 square feet (83.6 m^2) of suspended ceiling with lay-in tiles, and fire sprinkler piping with a riser pipe, a main run, and three sprinkler branch lines that included sprinkler drops as shown in Figure 6-7a and Figure 6-7b. Additional details of the sprinkler piping systems are available in Soroushian et al. (2016). In addition, two areas enclosed by self-standing partial height partition walls were designated as an office room at the fourth floor and a hospital room at the fifth floor and are shown in Figure 6-7c and Figure 6-7d, which were used to stage a variety of furniture and other unattached items (Guzman and Ryan, 2015b).



(a) (b) (c) (d)

Figure 6-7 Nonstructural components on the test building: (a) partial height interior partition wall and ceilings (Soroushian et al., 2016); (b) sprinkler drop (Soroushian et al., 2016); (c) fourth-floor hospital room (Guzman and Ryan, 2015a); and (d) fifth-floor office room (Guzman and Ryan, 2015a).

Test Protocol

The test building was subjected to a total of 41 sinusoidal and earthquake tests including 13 distinct earthquake records. The test schedule consisted of three days of testing for the TP configuration, two days of testing for the hybrid configuration, and one day of testing for the fixed-base configuration. The earthquake motions, which are mainly multicomponent acceleration histories recorded in previous earthquakes, were applied on the building tested as two-dimensional (horizontal directions only) and three-dimensional excitations, some with considerably large vertical components. Known records that were incorporated into the experimental program include El Centro Station (1940 El Centro earthquake), Tabas Station (1978 Tabas earthquake), Los Gatos Presentation Center (1989 Loma Prieta earthquake), Rinaldi Receiving Station and Sylmar - Olive View Hospital (1994 Northridge earthquake), Takatori and Kobe JMA (1995 Kobe earthquake), TCU080 Station (1999 Chi-chi, Taiwan earthquake), and Iwanuma Station (2011 Tohoku, Japan earthquake).

In each configuration, the three-dimensional Rinaldi Receiving Station (RRS) motion induced the largest drift and acceleration demands on the structure. Table 6-5 lists peak values of horizontal (PGA-xy) and vertical (PGA-z) accelerations recorded at the table level, peak floor accelerations (PFA) and vertical slab accelerations (PSA) at the fifth and roof levels, and the peak fourth and fifth story drifts. These locations were selected because they correspond to the location of the nonstructural components in the building. Although the horizontal input acceleration intensity applied to the fixed base configuration was considerably smaller than the isolation configurations, recorded floor accelerations were comparable in the three configurations. In addition, the same vertical acceleration intensity was applied to each configuration, so that recorded slab vibrations were also similar in the three configurations.

Table 6-5 Summary of Peak Acceleration and Drift Demands Recorded in Each Configuration

Response Quantity	TPB	Hybrid LRB	Fixed Base
PGA-xy (g)	1.21	1.15	0.41
PFA Fifth Floor (g)	0.90	0.67	1.19
PFA roof (g)	0.85	1.12	1.21
PGA-z (g)	1.24	1.26	1.06
PSA 5^{th} (g)	6.77	4.76	4.49
PSA roof (g)	6.44	7.03	6.11
Story Drift Fourth Floor (%)	0.33	0.42	0.66
Story Drift Fifth Floor (%)	0.31	0.44	0.49

Table 6-6 through Table 6-8 summarize the dynamic tests conducted in the three test configurations. Included for each motion are the test date, earthquake event and site (station where motion was recorded), and scale factors (corresponding to each of the three directions). Additional information about the realized ground motions as well as design of the test building and nonstructural components can be found in Ryan et al. (2013).

Table 6-6 Summary of Earthquake Test Motions in the Triple Pendulum Bearing Base Isolation Configuration (Ryan et al., 2013)

Date (dd/mm/yy)	Time	Duration (seconds)	Simulation Abbreviation	Motion	Scale Factor			Damage Inspection
					X	Y	Z	
17/08/11	12:01	41	SIN65(X)	Sine Wave	0.65	0.00	0.00	No
17/08/11	12:40	41	SIN100(X)	Sine Wave	1.00	0.00	0.00	No
17/08/11	13:42	41*	WSM80	Superstition Hills, Westmorland	0.80	0.80	0.80	No
17/08/11	14:30	41*	ELC130	El Centro, Imperial Valley	1.30	1.30	1.30	No
17/08/11	15:20	20*	RRS88	Northridge, Rinaldi Rec Station	0.88	0.88	0.88	Yes
17/08/11	17:16	41*	SYL100	Northridge, Sylmar	1.00	1.00	1.00	No
17/08/11	17:49	41*	TAB50	Tabas, Tabas Station	0.50	0.50	0.50	Yes
18/08/11	11:36	41*	LGP70	Loma Prieta Los Gatos Pres Ctr	0.70	0.70	0.70	No
18/08/11	12:26	82	TCU50(XY)	Chi-Chi, TCU	0.50	0.50	0.00	No
18/08/11	13:56	82	TCU70(XY)	Chi-Chi, TCU	0.70	0.70	0.00	No
18/08/11	14:32	196	IWA100(XY)	Tohoku, Iwanuma	1.00	1.00	0.00	No
18/08/11	15:46	327	SAN100(XY)	Sannomaru	1.00	1.00	0.00	No
18/08/11	16:35	41*	TAK100	Kobe, JR Takatori	1.00	1.00	1.00	No
18/08/11	17:05	41*	KJM100	Kobe, JMA Kobe	1.00	1.00	1.00	Yes
19/08/11	11:30	21	RRS88(XY)	Northridge, Rinaldi Rec Station	0.88	0.88	0.00	No
19/08/11	12:17	82	TCU80(XY)	Chi-Chi, TCU	0.80	0.80	0.00	No
19/08/11	13:08	41*	TAB80	Tabas, Tabas Station	0.80	0.80	0.80	No
19/08/11	14:02	41	TAB90(XY)	Tabas, Tabas Station	0.90	0.90	0.00	No
19/08/11	14:51	41	TAB100(XY)	Tabas, Tabas Station	1.00	1.00	0.00	No
19/08/11	15:28	82	SCT100(XY)	Michoacan SCT	1.00	1.00	0.00	No
19/08/11	16:19	41*	TAK115	Kobe, JR Takatori	1.15	1.15	1.00	Yes

* Complete ground motion is utilized in test

Table 6-7 Summary of Earthquake Test Motions in the Hybrid Isolation Configuration (Ryan et al., 2013)

Date (dd/mm/yy)	Time	Duration (seconds)	Simulation Abbreviation	Motion	Scale Factor			Damage Inspection
					X	*Y*	*Z*	
25/08/11	11:20	41*	WSM80	Superstition Hills, Westmorland	0.80	0.80	0.80	No
25/08/11	12:22	21	SIN100(Y)-1	Sine Wave	0.00	1.00	0.00	No
25/08/11	13:06	41*	VOG75-1	Vogtle #13	0.75	0.75	0.75	No
25/08/11	13:56	41*	VOG100	Vogtle #13	1.00	1.00	1.00	No
25/08/11	14:34	41*	VOG125	Vogtle #13	1.25	1.25	1.25	No
25/08/11	15:15	41*	VOG150	Vogtle #13	1.50	1.50	1.50	No
25/08/11	16:18	41*	VOG175	Vogtle #13	1.75	1.75	1.25	No
25/08/11	16:53	41*	DIA80	Diablo #15	0.80	0.80	0.80	Yes
26/08/11	12:03	41	DIA95(XY)	Diablo #15	0.95	0.95	0.00	No
26/08/11	12:49	41*	ELC130	El Centro, Imperial Valley	1.30	1.30	1.30	No
26/08/11	13:45	196	IWA100(XY)	Tohoku, Iwanuma	1.00	1.00	0.00	No
26/08/11	14:38	21	RRS88(XY)	Northridge, Rinaldi Rec Station	0.88	0.88	0.00	No
26/08/11	15:21	21*	RRS88	Northridge, Rinaldi Rec Station	0.88	0.88	0.88	No
26/08/11	16:15	41*	VOG75-2	Vogtle #13	0.75	0.75	0.75	No
26/08/11	16:59	21	SIN100(Y)-2	Sine Wave	0.00	1.00	0.00	Yes

* Complete ground motion is utilized in test

Table 6-8 Summary of Earthquake Test Motions in the Fixed Base Configuration (Ryan et al., 2013)

Date (dd/mm/yy)	Time	Duration (seconds)	Simulation Abbreviation	Motion	Scale Factor			Damage Inspection
					X	*Y*	*Z*	
31/08/11	10:20	40	WHT100(X)-1	White Noise	1.00	0.00	0.00	No
31/08/11	10:30	40	WHT100(Y)-1	White Noise	0.00	1.00	0.00	No
31/08/11	10:39	40	WHT100(Z)-1	White Noise	1.00	0.00	1.00	No
31/08/11	10:51	41*	WSM80	Superstition Hills, Westmorland	0.80	0.80	0.70	No
31/08/11	11:03	40	WHT100-1	White Noise	1.00	1.00	1.00	Yes
31/08/11	12:07	40	WHT100-2	White Noise	1.00	1.00	1.00	No
31/08/11	12:19	21*	RRS35(XY)	Northridge, Rinaldi Rec Station	0.35	0.35	0.00	No
31/08/11	12:28	40	WHT100-3	White Noise	1.00	1.00	1.00	Yes
31/08/11	13:38	40	WHT100-4	White Noise	1.00	1.00	1.00	No
31/08/11	13:51	21*	RRS35	Northridge, Rinaldi Rec Station	0.35	0.35	0.35	No
31/08/11	14:03	40	WHT100-5	White Noise	1.00	1.00	0.00	Yes
31/08/11	15:13	40	WHT100-6	White Noise	1.00	1.00	0.00	No
31/08/11	15:25	21*	RRS35(XY)88(Z)	Northridge, Rinaldi Rec Station	0.35	0.35	0.88	No
31/08/11	15:34	40	WHT100-7	White Noise	1.00	1.00	1.00	Yes
31/08/11	17:07	40	WHT100-8	White Noise	1.00	1.00	1.00	No
31/08/11	17:23	196*	IWA70(XY)	Tohoku, Iwanuma	0.70	0.70	0.00	No
31/08/11	17:35	40	WHT100(X)-2	White Noise	1.00	0.00	0.00	No

* Complete ground motion is utilized in test

Test Results

The damage states of the fire sprinkler system, dominated primarily by damage to the suspended ceiling system (panel fallout and grid damage), were qualitatively assessed by inspection of video footage during each earthquake simulation. The identified damage states of the fire sprinkler system were subsequently associated with the peak horizontal and vertical acceleration demands of the structure. Since the horizontal floor accelerations were constrained to relatively low levels during base isolation configuration tests, the damage ratings were more closely correlated to vertical slab acceleration than horizontal floor acceleration. The acceleration-sensitive fire sprinkler system component damage initiated at slab accelerations of approximately 2g. Damage was minimal for slab accelerations from 2 to 3g, moderate for slab accelerations from 3 to 5g, and extensive for slab accelerations exceeding 5g. However, it is noted that further evidence is needed to conclude that the vertical acceleration demands observed in the experiments were representative of realistic structural systems (Ryan et al., 2015).

The test program revealed several damage patterns of the nonstructural systems that were specifically associated with the vertical excitation. First, the ceiling system with seismic bracing (roof floor) experienced significant grid damage and panel fallout the unbraced ceiling system (fifth floor) experienced very little damage. In some cases, roof accelerations were slightly larger than fifth floor accelerations; however, the discrepancy in performance was attributed to the presence of compression posts. The presence of rigid compression posts caused the ceiling grid system to accelerate in sync with the slab at accelerations exceeding 4g (see Table 6-5), which caused the unattached panels to dislodge. Panel fallout seemed to trigger subsequent grid damage, and large areas of the grid were compromised. Meanwhile, the connection of the unbraced ceiling to the slab was restricted to loose hanger wires; allowing the ceiling grid system to "float" relative to the slab, which advantageously maintained the integrity of the grid members and the panels. Damage at ceiling boundaries was also observed; a similar damage pattern was observed in the UNR experiments and is described in Section 6.1.3.

Many instances of damage to ceiling panels was observed due to panel-sprinkler head interaction. Various gap configurations did little to mitigate the damage; however, such damage is generally limited to replacing a ceiling panel. However, loosening of piping connections (e.g., branch lines relative to the main run, relative branch line connections) caused permanent rotation of branch line pipes and the associated sprinkler drops. This damage state was also associated with the substantial vertical accelerations that were observed.

Finally, few of the conventional damage mechanisms to the partition walls were observed due to the relatively small drifts that were induced in the experiment. Large relative vertical accelerations between floor slabs induced diagonal and vertical cracks on gypsum wallboards. For slip track connections (studs unattached to the top tracks), the studs moved laterally (popped out) from the tracks. On bulkhead partitions, which consisted of open studs extending several feet below the enclosed wall, severe stud buckling was observed.

Horizontal and vertical acceleration amplification factors were evaluated based on accelerations recorded at the floor column level and in the components. Observed horizontal amplification factors were comparable to the design a_p factor applied in ASCE/SEI 7 Equation 13.3-1. However, observed vertical amplification factors were substantially larger than the code a_p factors, which suggests the code may be unconservative in predicting vertical design forces for nonstructural components.

Essentially, the code a_p does not include any allowance for vertical slab vibration relative to the column.

The physical damage of the building contents at the two upper floors were evaluated by inspection the video recordings of their movement during the earthquake tests. The damage states of the contents were defined qualitatively into five discrete levels based on specific criteria pertinent to the observed behavior of the room contents (Guzman and Ryan, 2015b). This study also indicates that peak floor velocity (PFV) is a more consistent damage predictor for the unattached building contents compared to peak floor acceleration (PFA).

Test Dataset

The response of the structure and nonstructural components was measured with 642, 482, and 387 channels of instrumentation for the three test configurations, respectively. Data collected in this research project are archived and publicly available within the following three online NEES datasets (three experiments):

- Project: "TIPS - Tools to Facilitate Widespread Use of Isolation and Protective Systems, a NEES/E-Defense Collaboration," Experiment-3: Full Scale 5-Story Building with Triple Pendulum Bearings at E-Defense [DOI: 10.4231/ D3X34MR7R];
- Project: "TIPS - Tools to Facilitate Widespread Use of Isolation and Protective Systems, a NEES/E-Defense Collaboration," Experiment-4: Full Scale 5-Story Building with LRC/CLB Isolation System at E-Defense [DOI: 10.4231/ D3SB3WZ43]; and
- Project: "TIPS - Tools to Facilitate Widespread Use of Isolation and Protective Systems, a NEES/E-Defense Collaboration," Experiment-5: Full Scale 5-Story Building in Fixed-Base Condition at E-Defense [DOI: 10.4231/ D3NP1WJ3P].

6.1.3 University of Nevada, Reno Two-Story Structure Tests

As part of the project NEESR-GC: Simulation of the Seismic Performance of Nonstructural System, a series of system-level large-scale shake table experiments were conducted at the UNR-NEES test facility from December 2012 to April 2013 (Rahmanishamsi et al., 2014a). The primary purpose of these experiments was to investigate the system-level response and failure mechanisms of interactive nonstructural systems, including interior gypsum partition walls, suspended ceilings, and fire sprinkler systems. In addition, two enclosed areas on the second floor were employed to stage various unattached building contents, simulating a hospital or office environment (Guzman and Ryan, 2015b).

Test Structure

The testbed structure was a two-story steel frame with two bays in the longitudinal direction and one bay in the transverse direction as shown in Figure 6-8a. The test structure had a plan dimension of 60 feet (18.3 m) by 11.5 feet (3.5 m) and a total height of approximately 25 feet (7.6 m). This test frame weighed about 115 kip (510 kN). The columns were made from 10-inch (257-mm) deep rolled I-sections with constant thickness of 0.6 inches (15.6 mm) over the floors. The primary transverse beams consisted of two back-to-back 6-inch (152-mm) by 6-inch (152-mm) by $^{1}/_{8}$-inch (13-mm) angles, which were bolted (idealized as pin connections) to the column as shown in Figure 6-8b. Lateral resistance was provided by X-shape braces made of two 8-inch (200-mm) by 8-inch (200-mm) by 2-inch (25-mm) angles. In contrast, the primary longitudinal beams were 16-inch (406-mm) deep rolled I-sections, which were connected to the columns via mechanical pin connection as shown in Figure 6-8c. The lateral loads in the longitudinal direction were resisted by two chevron buckling restrained braces (BRB) at each story. The test frame was designed to enable large floor accelerations and inter story drifts during the testing without damage to the test frame. Two sets of BRBs were used; stiff BRBs (linear system) induced large floor accelerations and flexible BRBs (nonlinear system) induced large story drifts. Additional information about the test structure can be found in Rahmanishamsi et al. (2014a).

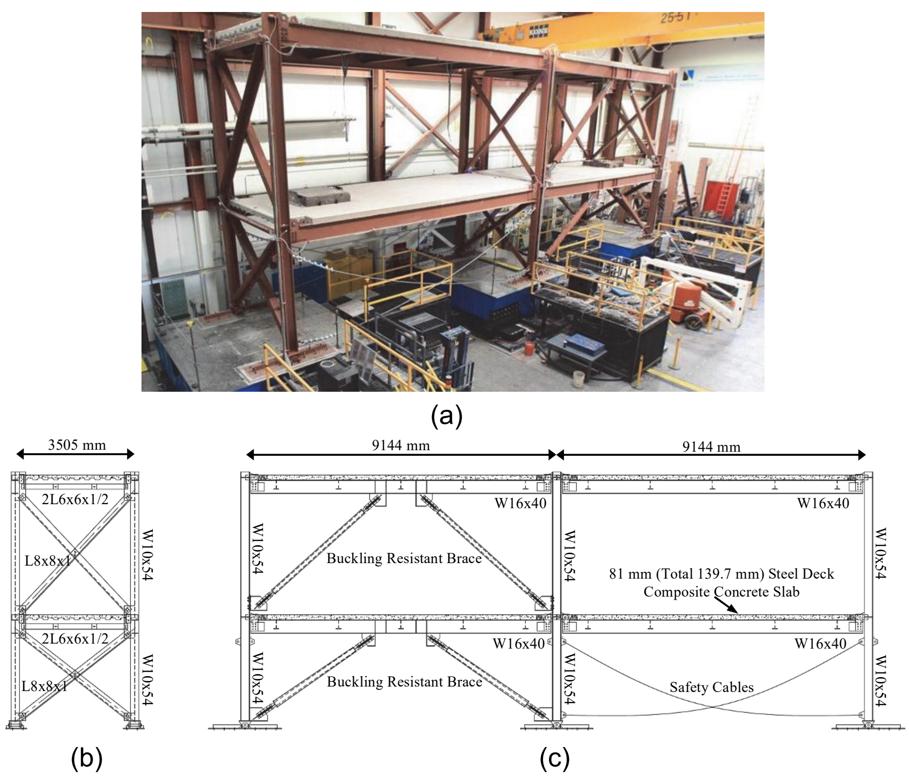


Figure 6-8 Two-story steel test structure: (a) photograph; (b) transverse elevation; and (c) longitudinal elevation (Rahmanishamsi et al., 2014a).

Nonstructural Components and Systems

Considering design variables from manufacturer catalogs and specific construction details, a total of fifteen configurations for suspended ceiling systems, two configurations for sprinkler piping systems, and fourteen configurations for partition walls were designed and tested in the shake table experiments. The ceiling configurations differed in terms of perimeter connection detail, bracing, ceiling area and ceiling weight. The partition walls were constructed using cold-formed light-gauged steel studs and tracks as well as gypsum boards. The variables in the wall configurations included the connectivity of the gypsum boards and studs to the top tracks, presence of return walls, details of wall intersections, height of the partition walls, and stud and track thickness. Two fire sprinkler piping systems were designed with different piping length using Schedule 40 steel pipes. A typical layout of the ceiling-partition-piping system installed on the test building is shown in Figure 6-9. Additional information about the test structure can be found in Rahmanishamsi et al. (2014a, 2014b).

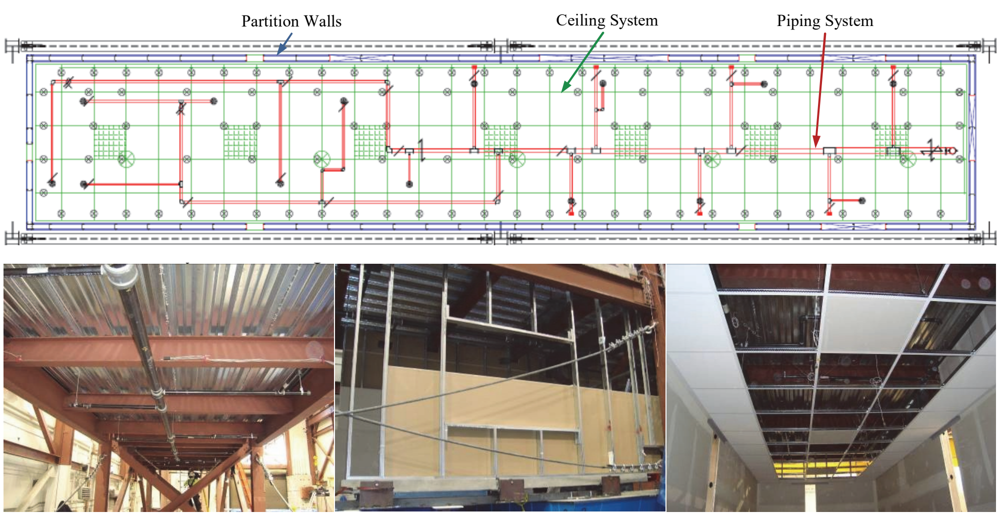


Figure 6-9 Ceiling-partition-piping system installed on the two-story steel test structure (Rahmanishamsi et al., 2014a).

Test Protocol and Achieved Motions

The system was subjected to a total of 59 uniaxial shake table motions (in parallel with the longitudinal direction of the test frame) in two test configurations: (1) with 42 motions applied on the linear structure; and (2) with 17 motions applied on the nonlinear structure. These shake table motions were artificially generated using an analytical procedure to achieve the target spectra on the desired levels. The targeted acceleration spectra study was developed following the AC156 spectrum (with A_{RIG_H} = 2.0g, A_{FLX_H} = 4.0g, S_{DS} = 2.5g, and a story height ratio $^{z}/_{h}$ of 0.5). For the linear structure, 12 of 42 motions aimed to match the target spectrum at the shake table (Unmodified-Linear), and 30 of 42 motions (Modified-Linear) were intended to match the target spectrum at the second floor. The motions for the nonlinear structure also aimed to match the target spectrum at the shake table level (Nonlinear).

The shake test program consisted of a total of 8 test sequences, each with a set of motions with progressively increasing amplitude. The linear test frame configuration was used in the first five test sequences and the nonlinear configuration was used in the remaining three test sequences. Maximum peak ground acceleration (PGAs) achieved on the shake table varied between 0.9 to 1.2g in the linear test sequences and 1.7 to 2.0g in the nonlinear sequences. In the linear test phase, the test frame achieved very large acceleration demands (PFA as large as 2.5g on the roof) while the interstory drift demands were relatively low (PIDR less than 0.75%). In the nonlinear test phase, the test frame experienced considerable large interstory drift demands (PIDR achieving 2.8% at the first level and greater than 2% at the second level), while the peak floor accelerations were relatively lower than those achieved in the linear tests due to the structural yielding (PFA between 1.0 to 1.5g).

The next sections summarize the response of various nonstructural components as observed in the experiments. Recent references include valuable quantitative statistical evaluations of response (Jenkins et al. 2016a; 2016b). For acceleration sensitive components (suspended ceilings and piping systems), acceleration amplification factors of the component relative to the testbed frame were evaluated in the study. For all components, fragility functions have been developed to indicate the statistical likelihood of various damage states as a function PFA or story drift, accounting for significant configuration variables.

Response of Cold-Formed Partition Walls

The response of partition walls in the experiments is summarized in Jenkins et al. (2016a). Observed damage to partition walls in these experiments that included interactions with the testbed frame, ceiling and piping system, was generally consistent with damage observed in component tests of partition walls. Although, debate has ensued over how to classify damage to partition walls. For instance, plastic hinge formation in the studs has previously been classified as Damage State (DS) 3, which could be interpreted as significant damage. However, upon initial inspection, the only indication of stud plastic hinging is screw popout, which could be repaired by simple reinforcement, and thus the hinging might not be apparent unless the walls were dismantled.

Three different connection details at the top of the walls were examined. In the full connection detail, the top track was connected to the deck by shot pins and to the studs with self-drilling screws. Since the connection is essentially fixed, the walls must accommodate the full story drift and experienced damage such as stud plastic hinging, screw popout, boundary stud damage, and separation of walls at the corners.

The slip track connection detail omits the track to stud connection and allows sliding of the studs relative to the top tracks. The sliding mechanism eliminated much of the damage along the wall in-plane, but damage at wall boundaries was still observed because out-of-plane walls constrained the sliding at the boundaries. Furthermore, the studs were observed to sometimes slide out of the top tracks. The sliding/frictional connection incorporated additional details to facilitate sliding between the top track and steel plates attached to the deck. The concept of this detail is to eliminate the boundary damage by allowing adjacent walls to slide together as a system. The detail improved the performance at wall boundaries, in particular preventing the separation of return and longitudinal walls. However, it was found that this detail was difficult to construct properly (such as providing sufficient gaps to allow full sliding everywhere) and the performance, particularly when subjected to large drifts, was not as good as previous component tests due to the system level interactions (Rahmanishamsi, 2016).

Various configurations of free standing and braced partial-height partition walls were tested. Unbraced partial-height partitions with no return walls were found to be highly vulnerable to seismic loads as a result of out-of-plane movement. Considered bracing mechanisms included: (1) 45-degree out-of-plane steel studs to connect the tops of the partition walls to the deck above; and (2) connection of tops of partition walls to the ceiling grid system, which was anchored to the deck using two out-of-plane 45-degree steel wires. The stud braced walls were also found to be vulnerable to failure of the stud connection. Vulnerability of partial height partitions should be highlighted, as they are found to be commonly used in practice (Rahmanishamsi, 2016).

Response of Suspended Ceilings

According to Rahmanishamsi (2016), the response of suspended ceiling systems in the experiments is evaluated in a manuscript that is currently under review. A variety of different ceiling configurations were tested. Some ceiling configurations included seismic bracing (steel stud compression posts and 45-degree oriented wire restrainers) while others were unbraced. Unlike the experiments at E-Defense presented in Section 6.1.2, the seismic bracing did not significantly influence the performance; however, the testbed was not subjected to vertical excitation and only mild vertical slab vibration was observed. Perimeter connections to adjacent partition walls were either pop-riveted with 2-inch perimeter angles or used seismic clips with $^{7}/_{8}$-inch perimeter angles. Generally, the grid system was fixed to the wall on two sides and “free” on two sides, using a $^{3}/_{4}$-inch clearance gap between the ceiling grid and perimeter angle. Ceiling manufacturers have advocated using seismic clips with a smaller perimeter angle to free up space. While the seismic clips did improve performance, the smaller perimeter angles were shown to be very prone to unseating of the grid member and return pounding that causes damage to one or more of the grid member, perimeter angle, and seismic clip, which is found to be consistent with observations from the E-Defense tests. According to the developed fragility functions, unseating occurs at an average PFA of about 0.25g for $^{7}/_{8}$-inch perimeter angles, and about 1.8g for 2-inch perimeter angles. Currently, special provisions are being introduced into ASCE/SEI 7-16 to address this issue. In addition, larger or heavier ceiling systems were found to be more vulnerable to the seismic excitation.

Response of Fire Sprinkler Piping Systems

The response of the piping systems in the experiments is evaluated in Jenkins et al. (2016b). Configuration variables such as continuous versus separate configuration, length of main pipes and branch lines, sprinkler head drop length, sprinkler head to ceiling panel boundary condition, hose type, and joint assembly, were considered in the design of the piping system. However, the same system was used for all experiments as a control for the ceiling system variations. Overwhelmingly, the fire sprinkler systems performed very well during the experiments, and the damage observed was less than that recorded from previous experiments. This may have been a result of experiencing only moderate accelerations from uniaxial excitations, or the fact that the system was small and pipe lengths were limited.

Consistent with prior experiments, damage to ceiling panels due to interaction with sprinkler heads was observed. Flexible hose drops substantially reduced the piping-ceiling interaction and thus led to less damage to the piping.

Various damage state descriptions for pipe joint rotations, axial forces in support elements, and pipe displacements were developed, and their occurrence observed. Subsequently, fragility functions were developed. Because joint failures and hanger failures were generally not observed in the experiments, the capacities of these elements were analytically calculated. Pipe displacement was found to be a governing fragility function because large pipe displacements induce contact and pounding with adjacent NCS due to system level interactions.

Test Dataset

The response of the structure and nonstructural components was measured with about 400 channels of instrumentation in a total of eight experiments. Data collected in this research project are archived and publicly available within the following online NEES datasets (in eight experiments):

- Project: “NEESR-GC: Simulation of the Seismic Performance of Nonstructural System,” Experiment-6: UNR System-level Shake Table Experiments on Acceleration Sensitive Nonstructural Components: Test Linear-1 through Experiment-13: UNR System-level Shake Table Experiments on Acceleration Sensitive Nonstructural Components: Test Nonlinear-3.

6.1.4 System-Level Shake Table Testing of Nonstructural Components Conducted at the E-Defense Facility, Japan

In the past decade, shake table testing of several full-scale buildings has been undertaken at the E-Defense facility in Japan. These projects largely focused on evaluating the seismic performance of particular building structural systems, including variants of base isolation technologies. However, in several instances nonstructural components were integrated within the test-building program. Data from these test programs may be more difficult to obtain and in many cases detailing is based on Japanese practice. Nonetheless, as these findings emerge and data become available observations should be harmonized with U.S. findings. Relevant projects that are documented in recent literature include:

- A four-story steel moment-resisting frame (SMRF) building that was tested in 2007 to investigate building collapse performance. Various nonstructural components, including autoclaved lightweight concrete cladding, glass windows, gypsum-sheathed interior partitions, and suspended ceilings, were incorporated into this test building. The seismic performance of the nonstructural systems in these tests is discussed in Matsuoka et al. (2008).
- A five-story steel moment-resisting frame (SMRF) building was tested in 2009, with the primary objective of investigating the influence of passive structural damping on SMRF buildings. As part of the research scope, three ceiling systems conforming to Japanese practice were tested within two floors on the east side of the building. Detailed discussions of the seismic behavior of the ceiling systems are available in Motoyui et al. (2010) and Hoehler et al. (2012).
- A series of full-scale shake table tests were conducted on a base-isolated four-story reinforced-concrete hospital building in 2008. The building was outfitted with a variety of furniture items, medical appliances, and service utilities to mimic a realistic hospital environment. The test results indicated that the base-isolated system effectively mitigated the damage to hospital components against near-fault ground motions, but dangerous movement of unanchored sliding or rolling equipment was observed during long-period ground, long duration motions. Further information from this test program is available in Sato et al. (2011).
- A four-story base-isolated RC building was tested under a series of three-dimensional ground motions in 2010. The building superstructure was the same as that tested by Sato et al. (2011). However, this test building was outfitted as a medical facility with a wide variety of hospital contents to investigate the effects of vertical response on building contents and nonstructural components. Although the rubber base isolation system significantly amplified vertical accelerations in some cases, the damage caused by the vertical motions was not

detrimental when peak vertical floor accelerations remained below 2g. Further information on this test program is available in Furukawa et al. (2013).

In addition, several recent projects are documented in publications written in Japanese:

- Experimental Study on the Seismic Loss of Functionality of Buildings due to Interior Space Damage: Part 1 Overview of Experiments (http://ci.nii.ac.jp/naid/110009654897);
- E-Defense Shake Table Tests of Existing Timber School Buildings: Part 6. Classroom Environment (http://ci.nii.ac.jp/naid/110009519390);
- A Study on Restraining Deformation of the Seismic-Isolated Layer of Houses Based on Shaking Table Tests of a Full-Scale House (https://www.jstage.jst.go.jp/article/aijs/79/699/79_565/_article/references/-char/ja/); and
- Collapse Mechanism of Wide-Area Suspended Ceiling System Based on E-Defense Full-Scale Shake Table Experiments (http://dil-opac.bosai.go.jp/publication/nied_tech_note/pdf/n391_a.pdf).

6.2 Component Testing

This section summarizes a variety of component-level experimental studies of major nonstructural systems conducted within the past decade in the United States. These studies include both pseudo-static loading tests of drift-sensitive systems (e.g., stairs, cold-formed steel partition walls) as well as shake table testing of acceleration-sensitive systems (e.g., suspended ceilings, building contents) and nonstructural anchored systems.

6.2.1 Egress Systems

Egress systems, such as stairs and elevators, are critical NCSs in a building or other structure, as they must remain operational for immediate access to a building following an earthquake or other disaster. Despite their significance, little attention has been dedicated to the understanding of seismic behavior of egress systems. Only one experimental study has focused on stair systems (Higgins, 2009), while the component-level experimental studies of elevator systems remain essentially non-existent.

Higgins (2009) tested two full-size prefabricated steel stair assemblies using a cyclic pseudo-static displacement loading protocol. The two specimens employed similar structural system and details but different stair treads. The specimens were configured as scissors stairs with two flights and an intermediate landing located mid-height as shown in Figure 6-10a. The story height of each specimen was 12 feet (3.6 m), and the landing dimension was about 4 feet (1.2 m) by 8 feet (2.4 m).

Lateral displacements were successively applied at the top landing in the two horizontal directions, while the lower flight and the base of the landing were bolted to the base frame.

The imposed lateral displacement cycles were applied with progressively increasing amplitudes up to the attainment of maximum target displacement corresponding to an interstory drift ratio of 2.5%, which is representative of the seismic design demands of moment frame buildings in the United States. Following the completion of the simulated seismic lateral displacement loading, the specimens were subjected to extra lateral displacement loading tests considering a full-factored vertical loading condition to assess their post-earthquake vertical loading capacity. At the completion of the tests, the two specimens remained operable with only moderate damage in the form of connection yielding and initial cracking as shown in Figure 6.10b and Figure 6.10c (Higgins, 2009).

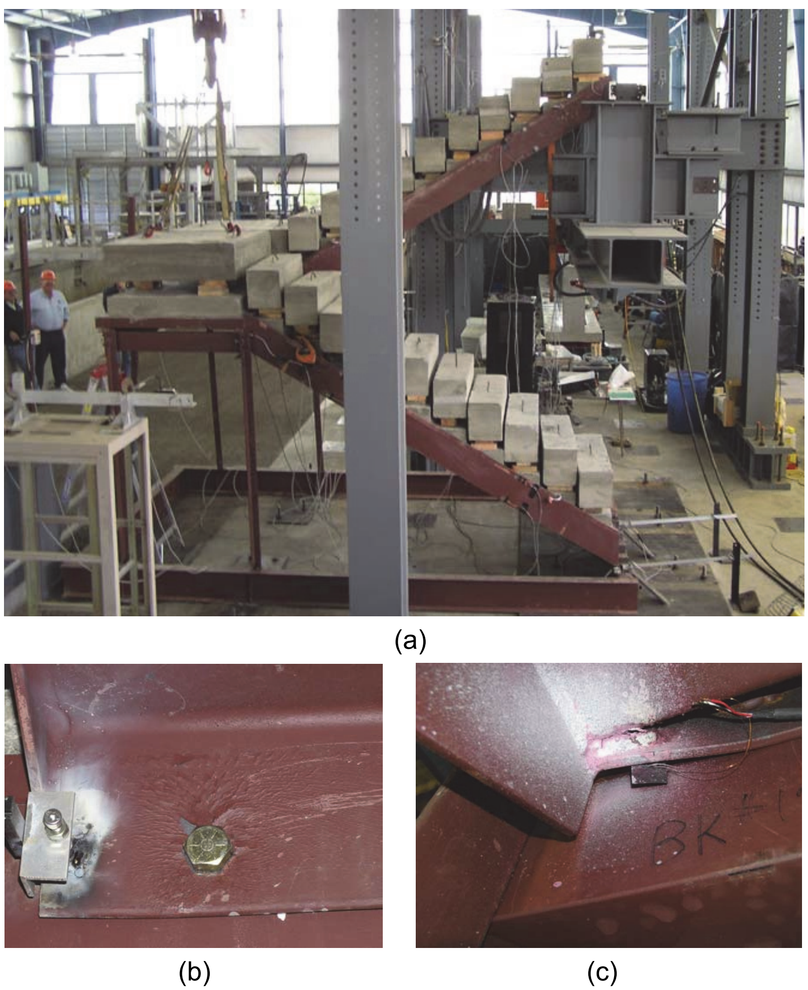
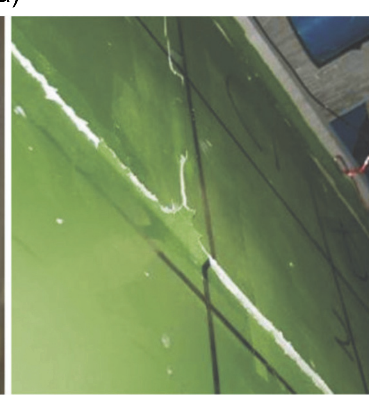


Figure 6-10 Experimental study of prefabricated steel stairs: (a) test specimen and test setup; (b) flight-to-landing connection yielding; and (c) initial weld cracking and connection plate buckling (Higgins, 2009).

6.2.2 Cold-Formed Steel Partition Walls

Several experimental studies have been recently conducted to understand the in-plane seismic behavior of CFS partition walls under quasi-static displacement loading. Restrepo and Bersofsky (2011) studied the seismic performance of eight pairs of nearly identical partition wall specimens, shown in Figure 6-11a and constructed following standard practices common to the United States. They found that partition wall damage causing temporary downtime of a building initiated at the interstory drift ratios as low as 0.5%, much lower than the level that structural damage would occur. To complement the work of Restrepo and Bersofsky (2011), Restrepo and Lang (2011) tested two identical specimens with different loading protocols and revealed that loading protocols only have limited effects on the seismic performance of the partition walls. Davies et al. (2011) conducted quasi-static and dynamic in-plane testing on 36 partition walls with different detailing or wall configurations. An example wall configuration is shown in Figure 6-11b. They studied the failure mechanism of these wall specimens and conducted seismic fragility analysis based on the test results. The drift levels associated with various partition wall damage states are consistent with those found by Restrepo and Bersofsky (2011).

(a)

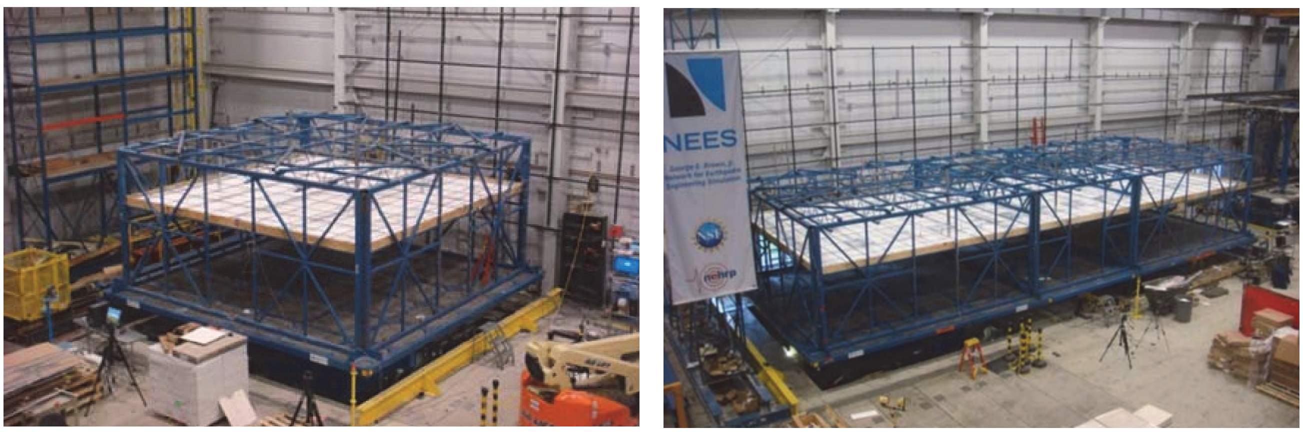


(b) (c)

Figure 6-11 Experimental study of cold-formed steel partition walls: (a) damaged wall specimen (Restrepo and Bersofsky, 2011); (b) metal framing damage; and (c) wall finish damage (Davies et al., 2011).

6.2.3 Suspended Ceilings

A series of shake table experiments to study the seismic behavior of suspended ceiling systems were conducted using a shake table array at the Structural Engineering and Earthquake Simulation laboratory (SEESL) at the University at Buffalo (UB). In the shake table test program, a total of fifteen suspended ceiling assemblies with different test configurations were tested: (1) five assemblies were tested on 20-foot (6.1-m) by 20-foot (6.1-m) test frame as shown in Figure 6-12a; and (2) ten assemblies were tested on 50-foot (15.2-m) by 20-foot (6.1-m) test frame as shown in Figure 6-12b. The main objective of this test program was to comprehensively investigate the component-level seismic performance of the suspended ceiling systems. The variables considered in the tests included: (1) uniaxial versus multi-axial excitations; (2) panel weights; (3) size of ceiling area; and (4) the effect of protective systems (bracing).



(a) (b)

Figure 6-12 Suspended ceiling shake table tests: (a) 20-foot by 20-foot test frame; and (b) 50-foot by 20-foot test frame (Ryu et al., 2012).

In each of the fifteen test configurations, the suspended ceiling was subjected to a series of test motions with increasing amplitude. The motions were intended to target a floor response spectrum at the roof level of the test frame, which followed the required response spectrum (RRS) as defined in AC156, *Acceptance Criteria for Seismic Certification by Shake-Table Testing of Nonstructural Components* (ICC ES, 2014). The peak vector sum of the horizontal table acceleration varied from 0.16 to 2.56g, which resulted in a maximum of 3.40g for the horizontal accelerations at the center of the roof level (PFA). The maximum vertical table acceleration was 0.68g, which resulted in a vertical acceleration of 1.54g in the mid-bay of the roof level. In each test, damage to the suspended ceiling systems occurred in the form of panel tile damage (e.g., dislodged panels, fallen panels) and ceiling grid connection damage (e.g., seismic clip failure, grid connection failure). The occurrence of specific damage patterns was dependent on the variables for each test configuration. Further details on the test program and test results can be found in Ryu et al. (2012).

6.2.4 Floor-Mounted Equipment and Furniture

In 2015, a research project to test current code-compliant seismic restraint systems for floor-mounted equipment and furniture was conducted under a grant by the Structural Engineers Association of Northern California (SEAONC) with support from the Pacific Earthquake Engineering Research (PEER) Center. This testing was conducted on the one-dimensional shake table at the Richmond Field Station of the University of California, Berkeley. The purpose of the project was to understand if commonly used seismic restraints meet code expectations for life safety and position retention and to determine the level of shaking that corresponds to failure for these designs.

Freezers/refrigerators were chosen as the focus of the test program for several reasons: (1) in hospital and laboratory settings, their contents are often of great importance or financial value; (2) they are representative of a wide spectrum of nonstructural components with similar properties; and (3) many popular details for freezers/refrigerators are complex to structurally analyze due to the use of adhesives, straps, and chains. A total of twelve refrigerators with various sizes and details were tested, with weights (including contents) ranging from 400 to 900 pounds. These included five slab-mounted, five wall-mounted, and two combination wall and slab-mounted units. In addition, five bookshelves were also included in the scope of the testing, since bookshelves with a large height-to-depth ratio are prone to overturning in earthquakes. Each bookshelf was 5.5 feet tall and weighed 330 pounds (including books). Some seismic restraint details required fastening the unit to a wall, thus requiring gypsum-sheathed stud walls to allow for testing of this type of detail. A 4-inch thick concrete slab was installed on the table to allow for anchorage into the concrete. The nonstructural component test setup is presented in Figure 6-13.

(a) (b)

Figure 6-13 Test setup of the nonstructural components: (a) freezers/refrigerators; and (b) bookshelves.

Test Protocol

Most of the tested seismic restraint details were designed to meet the ASCE/SEI 7-10 Chapter 13 requirements for essential (I_p = 1.5) nonstructural components. The design criteria were associated with an installation at mid-height ($^z/_h$ = 0.5) in a 16-story medical sciences building in San Francisco (S_{DS} = 1.13g). Anchorage to concrete was designed using an overstrength factor (Ω_0) of 2.5. In addition to the code-designed seismic restraint details, two "off-the-shelf" seismic restraints were tested, such as narrow clip angles screwed to metal studs at the top of the refrigerator or bookshelf. The purpose of this was to understand the efficacy of non-engineered solutions to meet seismic performance objectives. Each set of anchorage details was applied to the components and tested under five table motions in succession with increasing intensity. Qualitative data was collected on performance of the restraint systems after each test; however, repairs were not made between tests. The table motions were generated from a structural model of the building and corresponded to floor accelerations at mid-height of the building. The first four motions were developed from recorded motions that were adapted to the site. These corresponded to return periods of 225 years, 475 years (Design Earthquake), 975 years, and 2475 years (Maximum Considered Earthquake). The fifth motion was spectrally matched to the AC156 response spectrum for a return period of 975 years. Illustrated in Figure 6-14 is the floor response spectra of the input floor accelerations used in this test program.

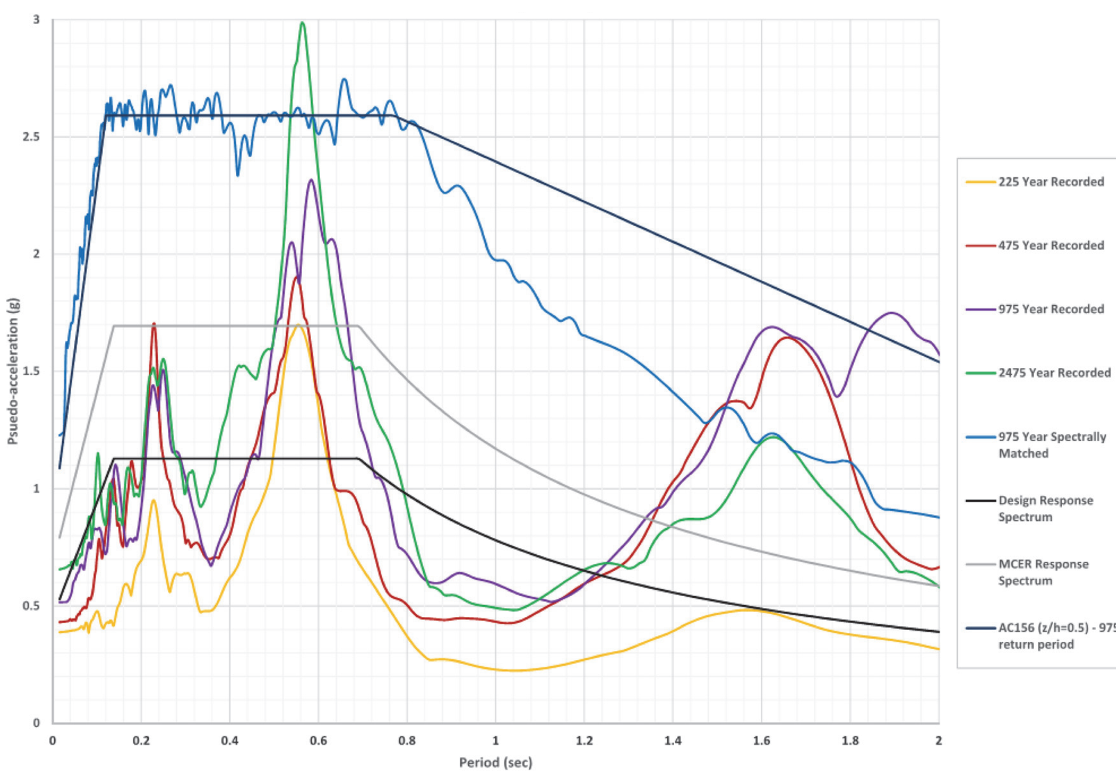


Figure 6-14 Floor response spectra of the input floor accelerations.

As testing was conducted on a unidirectional shake table, each anchorage design was tested in at least two orientations relative to the shaking (i.e., parallel to shaking and perpendicular to shaking). Where feasible, details were also tested at a 45-degree angle relative to shaking. The shake table was only capable of moving in the horizontal directions with the lack of vertical motion.

Test Results

Performance expectations for critical components (I_p = 1.5) designed in accordance with ASCE/SEI 7-10 are that they will "remain in place, sustain limited damage, and, when necessary, function following an earthquake" when subjected to Design Basis Earthquake (475-year return period) shaking. Due to project constraints, performance measures in the project were thus limited to position retention and level of damage, and the component functionality was not considered in the testing.

All properly installed details, including the "off the shelf" seismic restraint systems, met these performance objectives for the Design Earthquake (DE) motion, and performance for the majority of details significantly exceeded code expectations. Many details had observable damage following the Maximum Considered Earthquake (2475-year return period) and 975-year spectrally matched motions, but the damage could still be classified as "limited". Several trends in the performance of particular elements were observed during the testing:

- Expansion anchors exhibited good performance. No failures were observed.
- Details involving anchorage to metal stud walls exhibited good performance. Observable damage was limited to local deformation at the screws connecting the equipment to the wall. No failure of shot pins to the concrete slab was observed. Note that only 18-gauge, 8.5-foot tall studs were tested. Further testing is recommended with taller and thinner gauge studs.
- Refrigerator/freezer sheet metal to which the anchors were attached consistently exhibited poor performance. Throughout testing, the most commonly observed form of damage was tearing of the sheet metal where screws penetrated the refrigerator. Tearing of the sheet metal made the screws susceptible to loosening and in two tests the screws popped out. Damage to refrigerator sheet metal was observed in several restraint designs that required screwing into the equipment and on several refrigerator/freezer models.
- Products that use adhesives to attach the seismic restraint system to the refrigerator/freezer consistently exhibited good performance. No failures of the adhesive were observed. Damage to the sheet metal was also avoided as no screw penetrations were required for these systems.

- Improperly installed details performed poorly. In one test, clip angles were screwed into the gypsum board rather than the studs. This was the only anchorage scheme to fail (overturn) in the Design Earthquake (DE).

6.2.5 Shake Table Testing of Nonstructural Anchored Systems

Past earthquakes have repeatedly highlighted the seismic venerability of nonstructural components and their anchorage. To systematically understand the impact of anchorage to cracked concrete, a comprehensive shake table test project was conducted at UCSD. This effort focused on post-installed anchors and utilized model nonstructural components (Watkins and Hutchinson, 2011). The shake table tests focused on *floor-mounted* model nonstructural components anchored using different types of post-installed anchors (e.g., epoxy, expansion, drop-in, and undercut anchors), and, in particular, incorporated the effect of dynamic cyclic cracking on the anchor and component response.

As shown in Figure 6-15, a Weighted Anchor Loading Laboratory Equipment (WALLE) was constructed of steel tube and angle sections, and was designed to transfer the seismic loads of the nonstructural components to 4-single anchor Shear and Axial Measurement Units (SAMUs) placed at its base. The natural frequency of the loading equipment as well as the inertial load transferred to the anchors could be modified by varying the mass at the top of equipment. Two configurations were used during the testing, stiff and flexible, with natural periods of 0.10 and 0.25 seconds, which corresponded to a weight of 860 pounds (3,825 N) and 2,550 pounds (11,400 N), respectively.

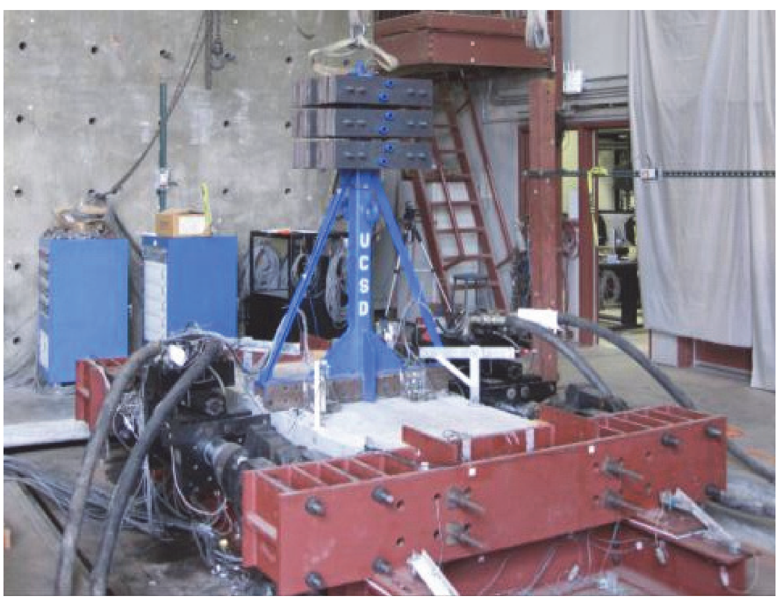


Figure 6-15 Shake table test setup of the anchorage behavior of floor-mounted nonstructural components (Watkins and Hutchinson, 2011).

The dynamic loading protocol included a total of 188 shake table tests, with three primary areas of investigation:

- System identification tests to characterize the dynamic properties of model nonstructural components and systems (NCSs) in terms of natural period and damping in both uncracked and cyclically cracked concrete.
- Correlation tests to determine the relationship between amplitude of anchor loading and corresponding crack width. The correlation tests were further divided into variable-phase correlation and in-phase correlation tests.
- Failure tests performed on four anchor types in cyclically cracked concrete. Anchor types included epoxy, torque controlled expansion, drop-in, and undercut post installed anchors.

This research project provided two important findings regarding the seismic behavior of anchors: (1) the maximum anchor load achieved during seismic loading of nonstructural components anchored in cyclically cracked concrete was generally in the range of 100 to 120% mean anchor ultimate tension load obtained from monotonic reference tests from an anchor installed in a static crack held open to a specified width; and (2) for floor-mounted, tension load cycling dominated anchored components that remain elastic during a seismic event, there is a direct correlation between the mean anchor displacement capacity in cracked concrete and the amplitude of seismic input motion that the anchored component system can withstand. Detailed discussion of the test program and the results are available in Watkins and Hutchinson (2011). Companion tests of suspended nonstructural components are reported by Mahrenholtz et. al (2014).

6.3 Designated Seismic System Qualification Testing

Special certification was introduced to provide a greater assurance that designated nonstructural components will perform as expected at design level seismic motions. Expectations for special seismic certified components is that the equipment will maintain structural integrity with minor yielding and damage allowed; however, the equipment must retain its functionality/operability following the design earthquake. Seismic qualification of equipment has been prevalent in the nuclear and defense industries since the 1970s, however, Special Seismic Certification of designated seismic systems was introduced in the commercial building code for designated seismic systems over a progression of years as follows. Many agree that it first became an enforceable requirement in the 2000 *International Building Code* (*IBC)* (ICC, 2000).

For buildings, the concept and verbiage of seismic certification was first introduced in ATC-3-06 (ATC, 1984). Section 1.6.3, which recommended testing or analyzing components and their anchorage, indicated that a special inspector shall be responsible for verifying that the special test requirements are performed by an approved testing agency for specified types of work in designated seismic systems. Within this section, Paragraph 1.6.3(E) for mechanical and electrical equipment stated that:

> "For Designated Seismic Systems or components requiring S or G performance ratings in Chapter 8, each component manufacturer shall test or analyze the component and its mounting system or anchorage as required in Chapter 8. A certificate of compliance for review and acceptance by the person responsible for the design of the Designated Seismic System and for approval by the Regulatory Agency shall be submitted. The basis of certification required in Section 8.3.4 shall be actual test on a shaking table, by three-dimensional shock tests, or by an analytical method using dynamic characteristics and the forces from Formula 8-2, or by more rigorous analysis providing for equivalent safety. The Special Inspector shall examine the Designated Seismic System component and shall determine whether its anchorages and label conform with the certificate of compliance."

The concept of seismic certification as well as inspection and labeling of equipment was continued in the 1985 *NEHRP Provisions* (FEMA, 1985). The language of Paragraph 1.6.3(E) in ATC-3-06 became essentially the same language in Paragraph 1.6.3.E of the 1985 *NEHRP Provisions*. In addition, Paragraph 1.6.5, Approved Manufacturer's Certification, in the 1985 *NEHRP Provisions* stated that:

> "Each manufacturer of equipment utilized in a building to be placed in Category E and a Designated Seismic System where the performance level required is noted in Chapter 8 as S or G shall be specifically approved by the Regulatory Agency and shall maintain an approved quality control program. Evidence of such approval shall be clearly and permanently marked on each component piece of equipment shipped to the job site."

Section 8.3.4 – Component Design in the 1985 *NEHRP Provisions* stated that:

> "When the direct attachment method is to be used for components with a performance characteristic level S or G in areas with a value of A_v greater than or equal to 0.15, the designer shall require certification from the manufacturer that the components will not sustain damage if subjected for forces equivalent to those resulting from Eq. 8-2."

> "When resilient mounting systems are used for components with performance criteria levels S and G, both the mounting systems and components shall require the certification stated above. Such systems shall be of the stable type."

> "Testing and certification shall be in accordance with the requirements of Section 1.6.3."

Special seismic certification was first introduced into the building code in the 2000 *IBC* within Section 1708.5 of Chapter 17, Section 1708.5. Many people agree that with the introduction of 2000 *IBC*, seismic certification of designated seismic systems first became an enforceable building code requirement. However, it was not enforced until the 2005 ASCE/SEI 7-05 was adopted by reference in the 2006 *IBC*. Therefore, few seismic certifications were performed between 2000 and 2006.

Section 1708.5 - Mechanical and Electrical Equipment of the 2000 *IBC* stated that:

> "Each manufacturer of designated seismic system components shall test or analyze the component and its mounting system or anchorage and shall submit a certificate of compliance for review and acceptance by the registered design professional in responsible charge of the design of the designated seismic system and for approval by the building official. The evidence of compliance shall be by actual test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance), or by more rigorous analysis providing for equivalent safety. The special inspector shall examine the designated seismic system and shall determine whether the anchorages and label conform with the evidence of compliance."

The seismic qualification requirement was retained in the 2006 *IBC* as Section 1708.5, Seismic Qualification of Mechanical and Electrical Equipment, with some clarifications as follows:

> "The registered design professional in responsible charge shall state the applicable seismic qualification requirements for designated seismic systems on the construction documents. Each manufacturer of designated seismic system components shall test or analyze the component and its mounting system or anchorage and submit a certificate of compliance for review and acceptance by the registered design professional in responsible charge of the design of the designated seismic system and for approval by the building official. Qualification shall be by an actual test on a shake table, by three-dimensional shock tests, by an analytical method using dynamic characteristics and forces, by the use of experience data (i.e., historical data demonstrating acceptable seismic performance) or by a more rigorous analysis providing for equivalent safety."

Detailed seismic certification requirements were issued in Chapter 13 of ASCE/SEI 7-05. The adoption by reference of ASCE/SEI 7-05 in the 2006 *IBC* and 2007

California Building Code (CBSC, 2007) and subsequent enforcement by OSHPD led to an industry-wide recognition of the requirement for seismic certification beginning in 2007. Other states have followed California's lead and special seismic certification is beginning to be enforced under the *IBC* in many other states.

Seismic certification may be conducted using several code accepted methods. Under Chapter 13 requirements in ASCE/SEI 7-05, seismic certifications can be conducted via analysis, testing, or experience data. Nonstructural components are grouped into two categories: (1) passive; and (2) active. Active components have either mechanical moving parts or energized electrical systems or a combination of both. In general, active components can be certified only by testing or experience data due to the complex nature of trying to predict operability of electrical circuits and moving parts in analysis software. Passive components can be certified by analysis, testing or experience data.

It has been important for standardization in the industry that a universally accepted test standard and acceptance criteria is used. AC156, *Acceptance Criteria for Seismic Certification by Shake-Table Testing of Nonstructural Components* (ICC ES, 2014) is an acceptance criteria and test protocol to establish the seismic capacity of a nonstructural component using a standardized acceleration response spectrum. The spectrum is based on a demand that uses S_{DS} and $^z/_h$ to establish control points A_{flx} and A_{rig} of the spectrum. These parameters can be conveniently related back to the seismic demand on a component in a building using S_{DS} and $^z/_h$ to be able to determine a demand to capacity ratio of the component.

During a shake table test performed to AC156, a component is first subjected to three directions of low level broadband or sine sweep input motions to determine the natural frequency of a component in its three orthogonal directions; front-to-back, side-to-side, and vertical. A transmissibility plot is generated to determine the fundamental natural frequency of the component in each direction. Transmissibility is the ratio of the response acceleration to the input acceleration as a function of frequency. As shown in Figure 6-16, the lowest peak of the plot defines the fundamental frequency of the component, in this case 2.83 Hz in the front-back direction.

Next the component is subjected to seismic motions. The seismic input motions are approximately 30-second long accelerations histories that have been preconditioned to fit the Required Response Spectrum (RRS). A sample of the acceleration time histories measured at the table level is shown in Figure 6-17. The spectra of the achieved test motions are called the Test Response Spectrum (TRS). The component being tested is referred to as the Unit Under Test (UUT). A sample of a RRS versus TRS spectra plot from an AC156 test is shown in Figure 6-18.

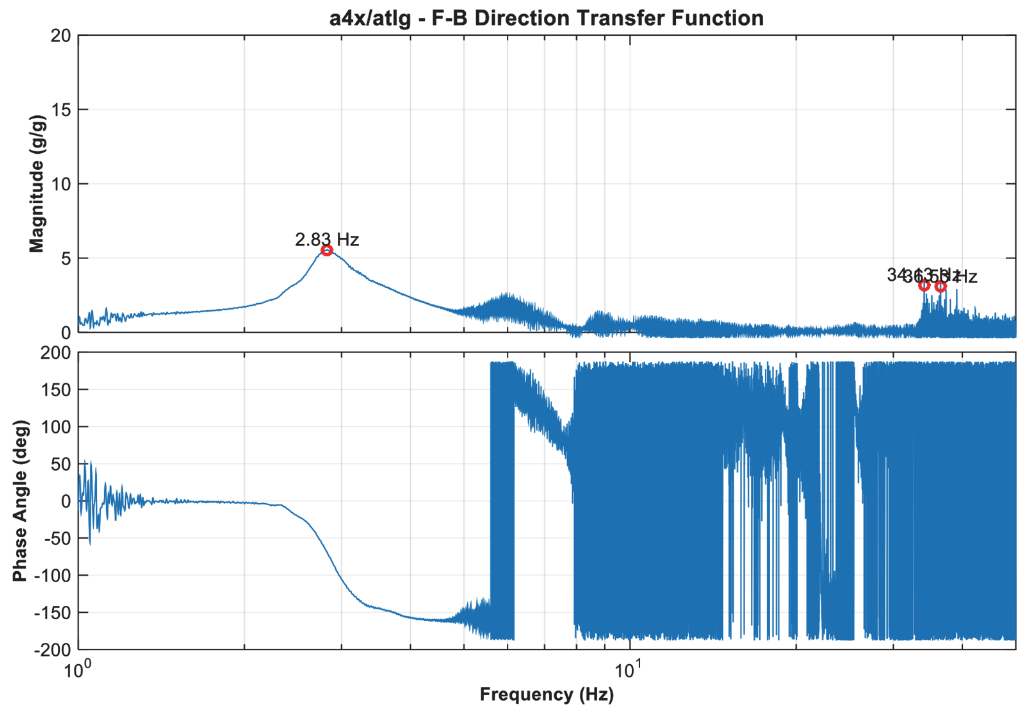


Figure 6-16 Transmissibility plot of natural frequency of a unit under test (UUT).

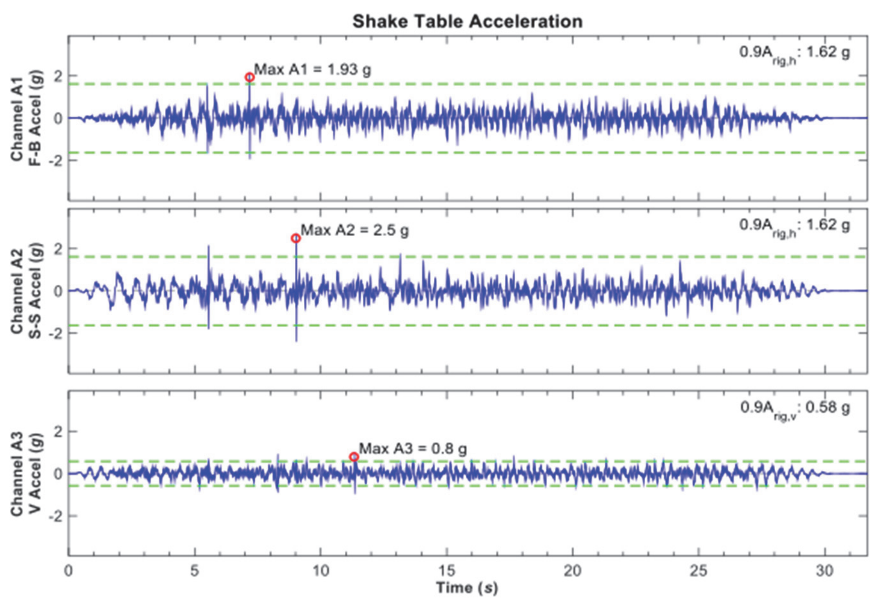


Figure 6-17 Acceleration time histories from an AC156 test.

The manufacturer and certification engineer select the Required Response Spectrum (RRS) based on geographic areas the component will be installed in or for project specific qualifications, the project site. ASCE/SEI 7-05 including Supplement No. 1 clarified that the maximum certification level required regardless of geographic location was for $S_{DS} = 2.0$g for equipment located on the roof of a building ($^z/_h = 1.0$) this translates to a peak spectral acceleration $A_{flx} = 3.2$g. The majority of certifications are conducted at this level due to its widespread applicability. After the

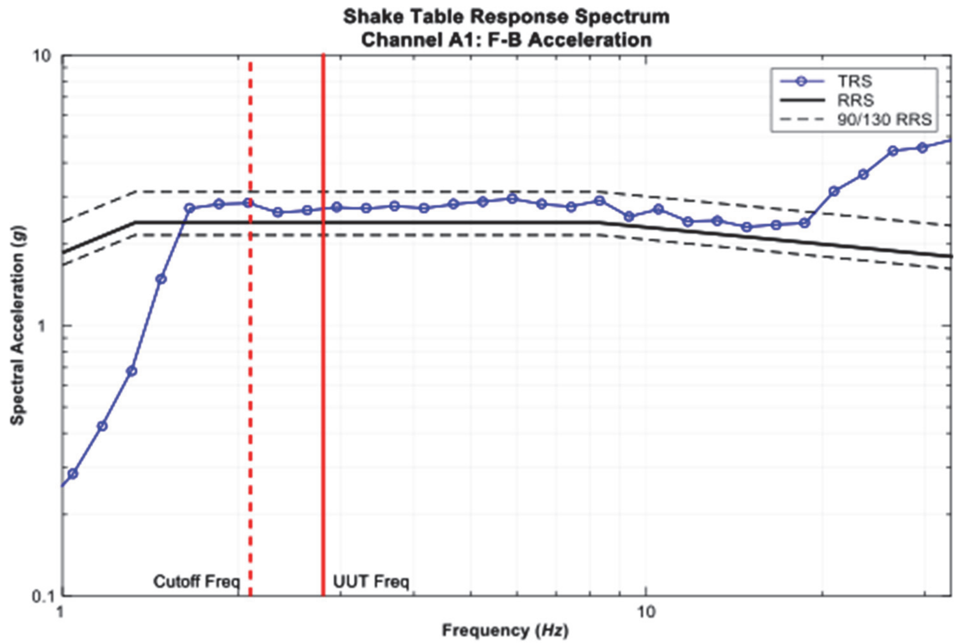


Figure 6-18 Required response spectrum (RRS) versus test response spectrum (TRS).

seismic test, the UUTs undergo operability testing to demonstrate that the equipment functions after an earthquake. The acceptance criterial for the functionality tests are established jointly between the manufacturer and the certifying engineer (e.g., fans operate, control panel performs electronic functions, and air handling unit operates).

A wide variety of mechanical, electrical and some architectural components have been tested. Testing has revealed vulnerabilities in the equipment and resulting changes have been made to improve the seismic ruggedness of nonstructural equipment. The test results have been positive for some types of robust/rugged equipment. The testing has also exposed vulnerabilities in some equipment, such as inadequate seismic load path bracing, screws used to resist out of plane loads in sheet metal, and anchorage. The largest amount of qualification testing in the commercial building industry has been performed for California hospitals. This is largely due to enforcement of code requirements by OSHPD. The OSHPD OSP program is a voluntary program were equipment manufacturers can have their equipment shake table tested per test standard AC156 and submit the test reports and product matrix data to OSHPD for pre-approval and listing on the public OSHPD website where they maintain a list of all pre-approved equipment at oshpd.ca.gov/fdd/pre-approval/ or more specifically at oshpd.ca.gov/fdd/pre-approval/specseiscert-wtemplate.html each approval is given an OSP number. OSHPD enforcement of the code requirements has dramatically changed the industry and has resulted in more reliable nonstructural components. However, certification is only typically performed on required designated seismic systems. In general, the types of equipment that have been tested are those required for OSHPD projects as defined in CBC Section 1705A. The following list is from approved changes for 2016 CBC, Section 1705A.13.3.1:

- emergency and standby power systems;
- internal communication servers and routers;
- elevator equipment (excluding elevator cabs);
- components with hazardous contents;
- exhaust and smoke control fans;
- switchgear and switchboards;
- motor control centers;
- fluoroscopy and x-ray equipment required for radiological/diagnostic imaging service (for service requirements see CBC Section 1224.18.1) and any fluoroscopy and/or radiographic system provided in support of diagnostic assessment of trauma injuries;
- CT (Computerized Tomography) systems used for diagnostic assessment of trauma injuries, except CT equipment used for treatment or in hybrid operating rooms, including those used for interventional CT, unless used for diagnostic assessment of trauma injuries;
- air conditioning units excluding Variable/Constant Air Volume (VAV/CAV) boxes up to 75 pounds;
- air handling units;
- chillers, including associated evaporators, and condensers;
- cooling towers;
- transformers;
- electrical substations;
- UPS and batteries;
- panelboards as defined in the California Electrical Code (CEC) Article 100;
- industrial control panels as defined in the CEC Article 100;
- power isolation and correction systems;
- motorized surgical lighting systems;
- motorized operating table systems;
- medical gas and vacuum systems;
- electrical busways as defined in UL 857; and

- electrical control panels powered by the life safety branch (CEC Article 517.32) or the critical branch (CEC Article 517.33).

One exception noted is that equipment and components weighing not more than 50 pounds supported directly on the structure (or surface mounted on equipment or components) are not required to have special seismic certification.

Seismic certification and resulting enforcement by OSHPD of the voluntary OSHPD OSP program has caused major improvements in the seismic ruggedness of certified components. It has also created a reliable means of establishing consistent seismic capacity per AC156 that can be compared to Chapter 13 of ASCE/SEI 7-05 seismic demands. There is currently an ASCE/SEI task force to further develop and refine the equipment certification testing protocol.

The structural community and standards committees have had some debate over what components are rigid and which ones are flexible as it is important for determining A_p amplification factors. The following histograms for nonstructural components and systems (NCS) natural frequency were developed based on test data available from shake table testing of equipment for the OSHPD OSP program. A database was compiled from test reports for OSP-0001-10 to OSP-0399-10 listed as of July 2014. Natural frequency data is extracted for the resonant frequency search testing performed under AC156 where low level sine sweeps or random broadband motion is input separately in the three primary axes of equipment Front-back, Side-Side, Vertical and natural frequency is extracted from the peak of the transmissibility plots. For more information on transmissibility, see AC156 or Institute of Electrical and Electronics Engineers (IEEE) standard IEEE 344 (IEEE, 2011).

The test data is representative of a wide variety of NCS which are required to remain operable in California hospitals per 2013 CBC, Section 1705A.12.4.1. There were 1,115 Mechanical UUTs (67%) and 540 Electrical units under test (UUTs) (33%) in the database that had frequency data. The isolated UUTs used spring isolators (not cork, rubber, or neoprene pads).

Typically, only the largest and smallest unit of a product family are required to be tested to obtain an OSHPD OSP preapproval therefore this data is representative of the extreme low and high frequencies of NCSs in these classes. Per AC156 each UUT is subjected to low level excitation resonant frequency searches in the Front-to-Back (FB), Side-to-Side (SS) and Vertical (V) directions. The lowest natural frequency from transmissibility plots is reported in the test report. The FB and SS frequencies can be different for a nonstructural component or system. For simplicity, all FB and SS data was lumped together and reported as horizontal natural frequency with the number of units (N) equal to two for one UUT since it has one FB frequency and one SS frequency. The horizontal natural frequency distribution of floor mounted components rigidly mounted to the floor is shown in Figure 6-19. Using a code definition of flexible (frequency less than 16.6 Hz), then 84% of equipment is flexible in the horizontal direction and 16% is rigid in the horizontal direction. The vertical natural frequency distribution of floor mounted components rigidly mounted to the floor is shown in Figure 6-20. Using a code definition of flexible, 46% of equipment is flexible in the horizontal direction and 54% is rigid in the horizontal direction. The horizontal natural frequency distribution of floor mounted components on spring isolators of various types from various manufacturers mounted to the floor is shown Figure 6-21. Using a code definition of flexible, 100% of equipment is flexible in the horizontal direction. The vertical natural frequency distribution of floor mounted components on spring isolators of various types from various manufacturers mounted to the floor is shown in Figure 6-22. Using a code definition of flexible, 100% of equipment is flexible in the horizontal direction.

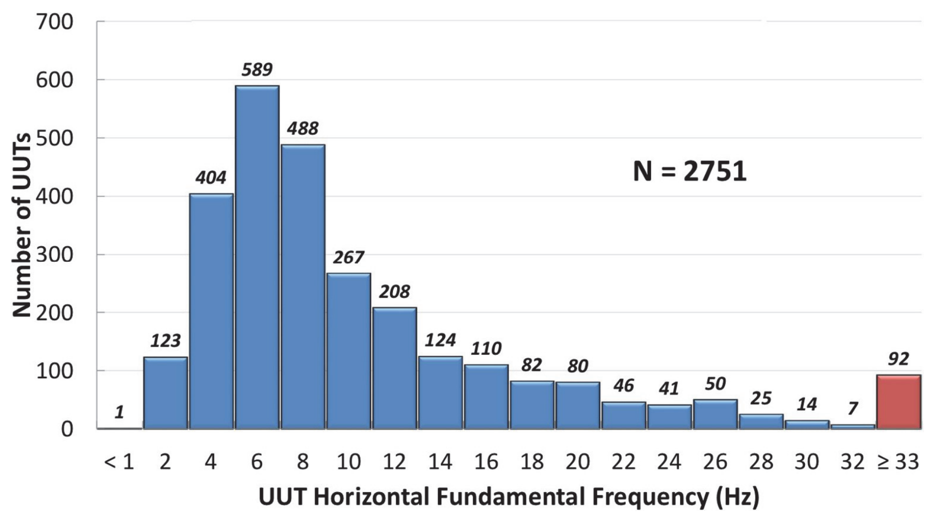


Figure 6-19 Horizontal natural frequency distribution of a unit under test (UUT) rigidly mounted to the floor.

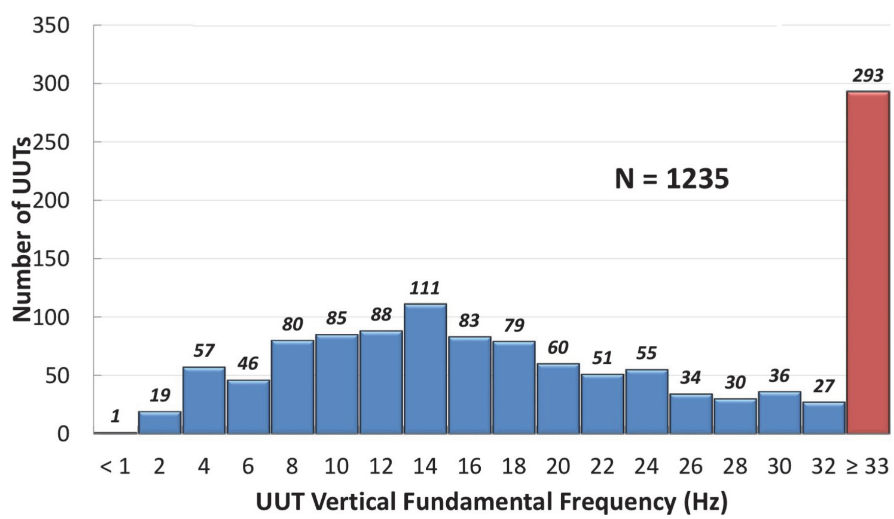


Figure 6-20 Vertical natural frequency distribution of a unit under test (UUT) rigidly mounted to the floor.

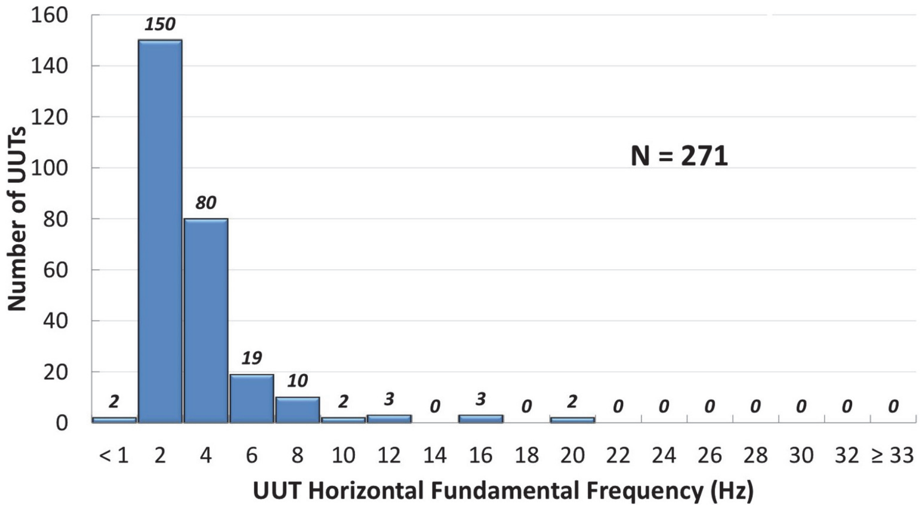


Figure 6-21 Horizontal natural frequency distribution of a unit under test (UUT) with spring isolators mounted to the floor.

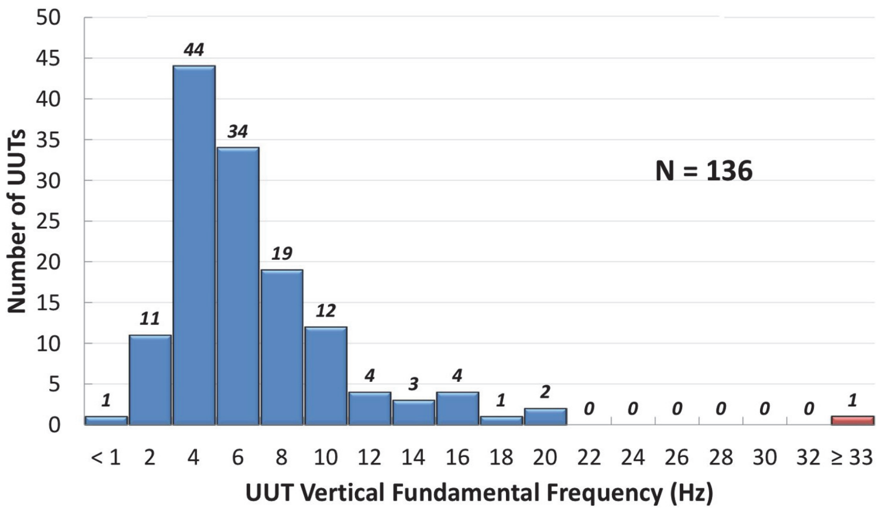


Figure 6-22 Vertical natural frequency distribution of a unit under test (UUT) with spring isolators mounted to the floor.

6.4 Seismic Response and Performance Database for Acceleration-Sensitive Nonstructural Components and Systems

As part of the ATC-120 Project, a spreadsheet database was developed for collecting the seismic response and performance of acceleration-sensitive NCSs during simulated seismic tests and field earthquakes. The measured data is essential for further studies on evaluating current seismic design code provisions for nonstructural systems as well as developing component-specific seismic fragility curves.

An excerpt from the spreadsheet database is presented in Figure 6-23. Within the table, data from an individual NCS associated with a specific test are organized row-by-row, while different data entries are contained in separate columns. Data entries in the spreadsheet database, from left-to-right, include metadata and test results in the following categories:

- Test program or earthquake: contains information about the test or earthquake short name (e.g., BNCS, Northridge), test type (e.g., system-level shake table test, component-level pseudo-static tests), test facility, and references.
- NCS metadata: contains general descriptions of NCS type/subtype (e.g., mechanical, electrical, and contents), geometry, weight, attachment details, mounting condition, and frequency/period.
- NCS design basis: design standard (e.g., ASCE/SEI 7-10), short-period design spectral acceleration S_{DS}, and design seismic force demand F_p.
- Target and measured response: measured acceleration of the input, building, NCSs, and floor response spectral values (e.g., PGA, PFA, $S_a(T)$, PCA, PFA/PGA, and PCA/PFA).
- Damage state: damage states of the component, support, and anchors (e.g., none, minor, moderate, or severe).

Currently the database contains test results of NCSs from one system-level experimental project and three component-level projects. In addition, field observations of the response of NCSs during the 1994 Northridge earthquake have been incorporated into the database.

	A	B	C	D	E	F	G	H	I	J	K	L	M
4	Test Program / Earthquake				NCS Type [1]	NCS Subtype	General Information						
5	Short Name	Ref(s)	Test type / Instrumented?	Test facility			Geometry [2]			Weight			Attach
6							L	W	H	Net Wt	Design Wt	Test Wt	Type
7							(in)	(in)	(in)	(lbf)	(lbf)	(lbf)	
8	BNCS	(1) - (2)	System-Level, shake table	NEES@UCSD	Content	TV cabinet	35.00	21.50	60.00	150	150	150	Attached to Wall w/ Nylon-adhesive Straps
9					Content	Kitchen Refrigerator	32.00	28.50	70.00	100	100	100	Unattached
10					Content	File Cabinet	36.00	19.00	53.50	180	180	180	Unattached
11					Content	Lab Cabinet	72.00	30.00	36.00	300	300	300	Screwed into Studs
12					Content	Cabinet	/	/	/	50	50	50	Screwed into Studs
13					Content	Table	/	/	/	50	50	50	Unattached
14					Content	Bookcase	24.00	12.00	60.00	70	70	70	Unattached
15					Content	Deli Refrigerator	58.00	34.00	83.00	610	610	610	Attached to wall w/ Rachet straps
16					Content	Wheeled Rack	47.25	18.00	60.00	65	65	65	Unattached
17					Content	Freezer	39.00	32.00	77.00				Nylon-adhesive Straps
18					Content	Cabinet	43.00	18.00	66.00	120	120	120	Attached to Wall-mounted Strut
19					Content	Kitchen Refrigerator	32.00	28.50	70.00	100	100	100	Unattached
20					Content	Deli Refrigerator	58.00	34.00	83.00	360	360	360	Expansion Anchors
21					Content	Freezer	39.00	32.00	77.00				Expansion Anchors
22					Content	Lab Cabinet	72.00	30.00	36.00	300	300	300	Screwed into Studs
23					Content	File Cabinet	36.00	19.00	53.50	100	100	100	Unattached
24					Content	Computer server	50.00	30.00	80.00	3000	3000	3000	Heavy duty anchor
25					Content	Computer server	30.00	50.00	80.00	3000	3000	3000	Heavy duty anchor
26													
27					Medical Equip.	Image Scanner							
28													
29													
30					Mech. Equip.	Cooling Tower	87.75	107.88	128.00	3500		6300	Spirng Isolators
31					Mech. Equip.	AHU	100.00	58.00	68.00	1500	1500	1500	Expansion Anchors
32					Architectural	Penthouse	240.00	120.00	120.00	3500	3500	3500	Expansion Anchors

Figure 6-23 Excerpt from a spreadsheet database on seismic response and performance of acceleration-sensitive nonstructural components and systems.

Chapter 7

Analytical Studies on Horizontal Floor Response Spectra

This chapter summarizes relevant findings from analytical studies related to acceleration-sensitive nonstructural components that can serve as foundation for future advancements on the estimation of horizontal acceleration demands. These acceleration demands are useful for calculating seismic design forces on supports and attachments of nonstructural components to the main structure. The objective is not to provide a comprehensive literature review on horizontal floor response spectra. Instead, this chapter incorporates information from selected studies to identify conditions and scenarios that result in consistent and significant underestimation or overestimation of component-acceleration demands. Ideally, this identification should be followed by problem-focused studies that will provide the required technical information to support appropriate modifications to code provisions.

The aforementioned evaluation should clearly identify instances in which simplified methods are unconservative or overconservative to guide the development of improved code provisions and establish ranges of applicability for these approaches. The consequence of this exercise would be that in some instances, the alternative approaches provided in ASCE/SEI 7-16, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2016) could be recommended for specific situations. Ideally, this assessment should be performed using high-fidelity models of structures and components in conjunction with experimental and experience data in order to improve the level of confidence on the aforementioned evaluation.

Many studies have evaluated individual parameters that form part of equations to estimate F_p. Results from a comprehensive evaluation of ASCE/SEI 7-16 Eq. 13.3-1 should help determine whether the departure point of simplified equations to estimate F_p when the dynamic properties of the component and the supporting structure are unknown should rely on: (1) the current approach that is based on individual parameters (e.g., in-structure and component amplifications); or (2) on a new approach to directly estimate peak component acceleration (PCA) demands (e.g., using generic floor response spectra from which appropriate estimates of the component design forces could be derived). These generic floor response spectra could vary with ground motion input spectra, height, building type, and importance. However, it is recognized that to be valuable for engineering, the fewer variables the better. A comprehensive evaluation ASCE/SEI 7-16 Eq. 13.3-4 should focus on whether modified correlations coefficients should be proposed when cross-correlation of modal responses is important. In all circumstances, it is important to also evaluate the ground motion intensity level at which assessments should be conducted.

7.1 Relationship between F_p and Floor Response Spectra

As stated in Chapter 2, seismic acceleration demands are estimated in the United States via the parameter, F_p, which is the horizontal equivalent static seismic design force acting at center of gravity of the component and distributed relative to the mass distribution of the component. Thus, F_p is directly proportional to component acceleration demands, and hence, can be used as a surrogate for component accelerations. This implies that the properties of F_p can be directly related to properties of floor response spectra if two parameters, the component importance factor, I_p, and the component response modification factor, R_p, are taken as 1.0.

The current estimation of F_p is influenced by the extent of available knowledge of the dynamic properties of the nonstructural component and the supporting structure. In this context, knowledge of dynamic properties of the component implies the combined component-support-attachment system. For instance, when these properties are unknown ASCE/SEI 7-16 Eq. 13-3.1 could be used. When dynamic properties can be estimated and sufficient information exists to develop adequate models, floor response spectrum methods and nonlinear response history procedures in Chapter 16, Chapter 17, and Chapter 18 of ASCE/SEI 7-16 can be utilized. For all methods, upper and lower limits corresponding to ASCE/SEI 7-16 Eq. 13-3.2 and ASCE/SEI 7-16 Eq. 13-3.3 are applicable.

The relationship between F_p and dynamic characteristics of the component and supporting structure is a critical issue. For instance, given a ground motion and a specific structure, it is well established in the literature that the ordinates of floor response spectra are strongly dependent on parameters and properties such as: ratios of component period to modal periods of the supporting structure, damping ratios, location of the component in the structure, type of lateral-load resisting system and distribution of stiffness along the height of the supporting structure, as well as the level of inelastic behavior of the component and the supporting structure. It is evident that any approach to estimate F_p (or floor response spectra) will not be as accurate for cases in which at least one of the aforementioned properties or parameters is not adequately characterized. However, it is extremely desirable from a manufacturer and practicing engineering perspective to have a generic F_p equation where the specific dynamic properties of the structure are not known or used.

7.2 Floor Response Spectrum Methods

Floor response spectra can be developed analytically using response history analysis or response spectrum approaches. Coupled models are used to develop floor response spectra for a given damping ratio by changing the period of the component during each response history analysis and recording the maximum response for the component versus vs period. In other words, each floor response spectrum ordinate will correspond to a different response history analysis using a coupled model.

Alternatively, a cascading approach in which the primary structure is analyzed first and its floor motions are used as input to evaluate the response of nonstructural components can be utilized. The cascading approach is appropriate when dynamic interaction effects can be neglected. An advantage of response history analyses is their applicability to both linear and nonlinear structures and components. However, response history analyses are more computationally intensive and time consuming.

Response spectrum techniques are adequate when the component and structure are linear elastic. Reasonable approximations can also be obtained when the component and/or the structure are mildly inelastic. A response spectrum approach that is based on the application of modal combination rules that account for modal cross-correlations is useful for estimating component acceleration demands as long as appropriate modal correlation coefficients are used. For instance, modal correlation coefficients for peak absolute floor accelerations are different from those conventionally used to estimate maximum relative response quantities, e.g., relative displacement demands (Taghavi and Miranda, 2006; Kumari and Gupta, 2007; Moschen et al., 2014). Modal combination rules such as the square root of the sum of the squares (SRSS) or the complete quadratic combination (CQC) used for the estimation of displacement demands cannot be directly used to estimate peak floor acceleration demands, since they do not take into account the correlation between ground acceleration and modal accelerations (Taghavi and Miranda, 2006). From this point of view, it is apparent that the application of ASCE/SEI 7-16 Eq. 13.3-4 when SRSS or CQC methods are used might need to be evaluated more thoroughly.

Many analytical studies have incorporated approaches based on simplified, and in some cases, empirical procedures to estimate floor response spectra. These procedures were developed using data obtained from analytical studies based on response history or response spectrum analysis that might or might not incorporate the nonlinear response of structures and components.

7.3 Relevant Characteristics and Quantification of Floor Response Spectra

In principle, floor response spectrum ordinates are maximum values of component accelerations that result from a filtering effect in which the ground motion at the base of the structure is filtered through the supporting structure. This behavior can be understood and explained analytically based on structural dynamics principles.

For components that can be classified as rigid, the result of this initial “filtering” process is floor motions that are harmonic in nature with frequencies that are mainly influenced by the modal properties of the supporting structure. The magnitude of the acceleration demands for rigid components depend on the dynamic properties of the supporting structure, as well as the frequency content and intensity of the ground motion. In general, rigid component acceleration demands tend to saturate, especially near the top of a supporting structure, as the structural system extends significantly into the inelastic range. The ratio of the resulting peak floor acceleration (PFA) to the peak ground acceleration (PGA) at any given height, PFA/PGA, is often referred to as the in-structure amplification factor, structure acceleration amplification factor, height factor, or absolute acceleration amplification factor.

For components that can be classified as flexible, amplification or reduction of maximum floor accelerations occurs when: (1) component frequencies are tuned to modal frequencies of the supporting structure; and (2) dynamic interaction effects are present between the component and the supporting structure (e.g., when the mass of the structural component is greater than about 1% of the mass of the supporting structure, Taghavi and Miranda, 2008). The ratio of PCA to the PFA, PCA/PFA, is generally denoted as component amplification factor, resonance factor, or dynamic amplification factor.

Analytical studies on PFA and PCA demands presented herein dealt with generic and building specific two-dimensional models of frame structures, structural wall systems, and dual systems exposed to simulated and recorded ground motion histories, as well as response and design ground motion spectra. These three seismic force-resisting systems are commonly used in earthquake regions of the United States. Relevant lessons learned from these studies are summarized next.

7.3.1 In-Structure Amplification and Peak Floor Accelerations

Several analytical studies have addressed the quantification of in-structure amplification via the estimation of PFA demands. At any given level, PFA represents the anchor point for floor response spectra, for it is the maximum acceleration of an infinitely rigid component attached to the structure. In the context of the estimation of F_p for rigid components, the most salient observations from these works are summarized below:

- Given an input ground motion and dynamic response characteristic for a specific structure, the distribution of PFA demands along the height of structures is significantly influenced by: (1) modal periods, especially the initial fundamental period of vibration, of the supporting structure; (2) type of lateral load resisting system or the lateral stiffness ratio (a parameter that quantifies the degree of participation of the global flexural and global shear deformations of the structure, and hence, defines the shape of lateral response) of the supporting structure; and (3) level of inelastic behavior of the supporting structure.

- PFA demands tend to increase with height for elastic short-period, first-mode dominated structures. As the fundamental period of the elastic supporting structure increases, PFA demands do not grow linearly with height and generally tend to fluctuate as shown in Figure 7-1 and Figure 7-2. Estimates of peak floor accelerations represented by ASCE/SEI 7-16 Eq. 13.3-1 increase linearly with height to a maximum value of three (3) at the roof level independently of the fundamental period of the supporting structure. For a specific structure a more accurate estimation of the variation of PFA with height can be obtained from the application of the alternative methods included in ASCE/SEI 7-16. However, the upper and lower limits of ASCE/SEI 7-16 Eq. 13.3-2 and ASCE/SEI 7-16 Eq. 13.3-3 still apply. In addition, equations incorporated in FEMA P-58, *Seismic Performance Assessment of Buildings* (FEMA, 2012a; 2012b; 2012c), to estimate PFA/PGA can also be applied. Improved FEMA P-58 equations will be published in the near future.

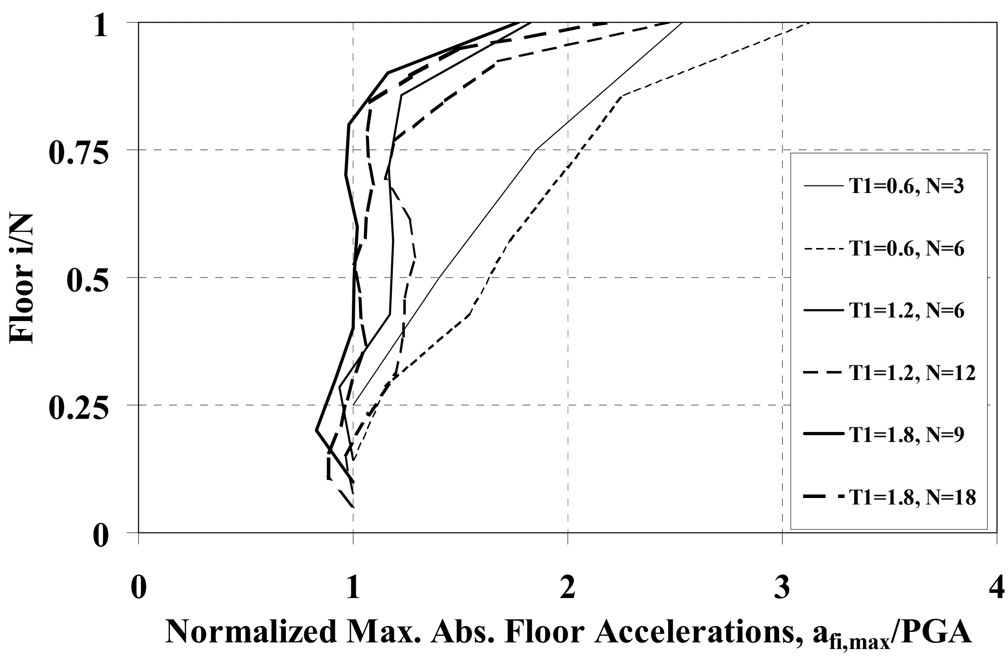


Figure 7-1 Normalized median peak floor acceleration (PFA/PGA) for elastic frame structures with different numbers of stories and fundamental periods (Medina and Krawinkler, 2004).

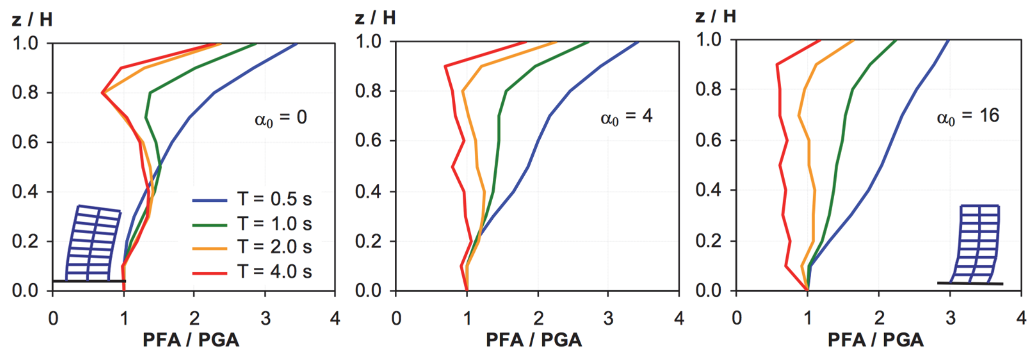


Figure 7-2 Variation of peak floor accelerations (PFA) along the height of buildings with different fundamental periods of vibration and different lateral stiffness ratios α_0 (Taghavi and Miranda, 2006).

- For a given fundamental period, moment-resisting frame buildings tend to have smaller roof accelerations than structural-wall buildings or braced-frame buildings that deflect primarily in flexure. The parameter $\alpha_0 = 0$ in Figure 7-2 corresponds to buildings that deform laterally like a flexural beam (e.g., shear wall systems), while $\alpha_0 = 20$ corresponds to buildings that deform laterally like a shear-beam (e.g., moment frame systems). The intermediate case of $\alpha_0 = 4$ corresponds to buildings with lateral deformations that are a combination of flexural and shear behavior. ASCE/SEI 7-16 estimates of peak floor accelerations based on ASCE/SEI 7-16 Eq. 13.3-1 do not consider this dependence on lateral load resisting system or lateral stiffness ratio. Improved estimates can be obtained by applying alternative procedures in ASCE/SEI 7-16 or PFA/PGA equations provided in FEMA P-58. As stated above, improved FEMA P-58 equations will be published in the near future.

- Inelastic behavior of the supporting structure causes a decrease in PFA demands with respect to the elastic case, as well as a variation of PFA demands along the height that is strongly dependent on the ground motion intensity level. For instance, as the ground motion intensity increases, the variation of PFA with height changes from PFA/PGA ratios that can be greater than one to ratios that are consistently smaller than one, regardless of fundamental period as shown in Figure 7-3 and Figure 7-4. PFA demands tend to saturate with an increase in ground motion (relative) intensity (η or RI) defined by the ratio of the normalized 5%-damped elastic spectral acceleration at the fundamental period of the structure to the base shear coefficient defined as $[S_a(T_1) / \mathrm{g}] / (V_b / W)$, where V_b is the base shear strength and W is the effective seismic weight, especially at the top floors (e.g., Medina and Krawinkler, 2004; Taghavi and Miranda, 2006; Sankaranarayanan, 2007; Medina and Clayton, 2010). In general, PFA in taller structures are the most affected by inelastic behavior (Taghavi and Miranda, 2006). The estimation of F_p based on ASCE/SEI 7-16 approaches, except for floor response spectra from nonlinear response history analysis, accounts generically and subjectively for both: (a) the influence of inelasticity in the supporting structure; and (b) period shifting of the component caused by its nonlinearity via the implementation of the upper limit of ASCE/SEI 7-16 Eq. 13.3-2.

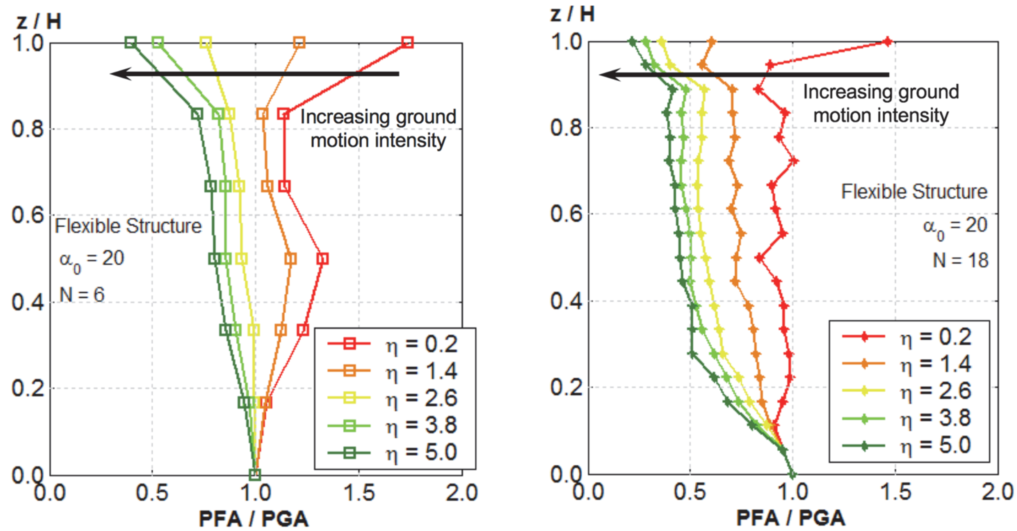
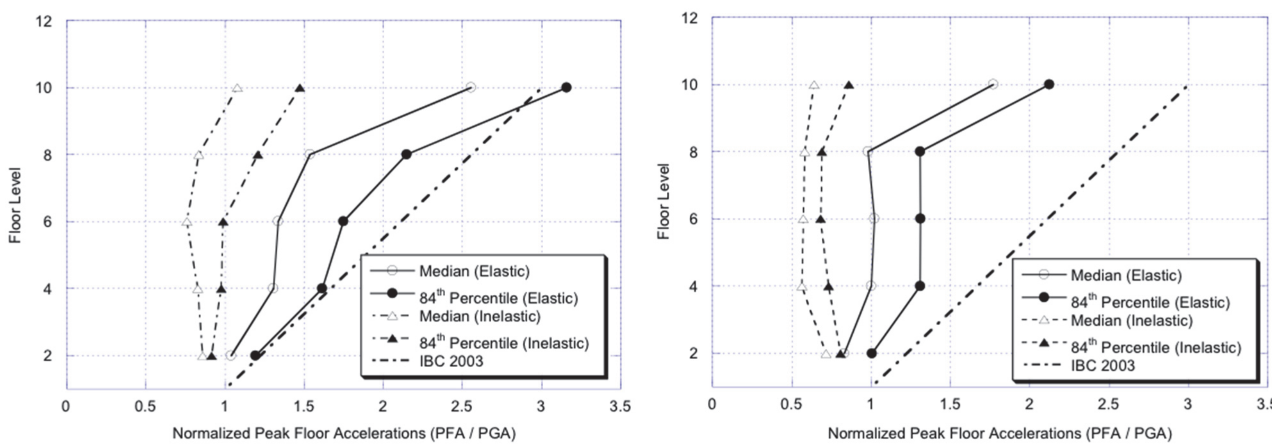


Figure 7-3 Normalized peak floor acceleration (PFA/PGA) versus height for mean responses from different intensity levels, η, for a 6-story (left) and an 18-story (right) flexible moment frame buildings (Miranda and Taghavi, 2009).



(a) 5%-damped, stiff building

(b) 5%-damped, flexible building

Figure 7-4 Normalized median and 84th percentile peak floor acceleration (PFA/PGA) ratios for elastic and inelastic responses of a 9-story building compared to 2003 *International Building Code* (Medina et al., 2006).

- On average, as shown in Figure 7-5, for frame buildings exposed to design-level ground motions, code estimates of PFA based on ASCE/SEI 7-16 Eq. 13.3-1 overestimate PFA demands at the top floor of some specific flexible buildings and underestimate PFA demands at the bottom floors of some specific rigid buildings (Chauduri and Hutchinson, 2004; Medina et al., 2006; Medina and Clayton, 2010; Singh et al., 2006; Sankaranarayanan, 2007; Miranda and Taghavi, 2009; Wieser et al., 2012; Wang et al., 2014). The aforementioned underestimation is also present, as shown in Figure 7-6, at the bottom of rigid structural wall buildings exposed to design-level ground motions (Medina and Clayton, 2010).

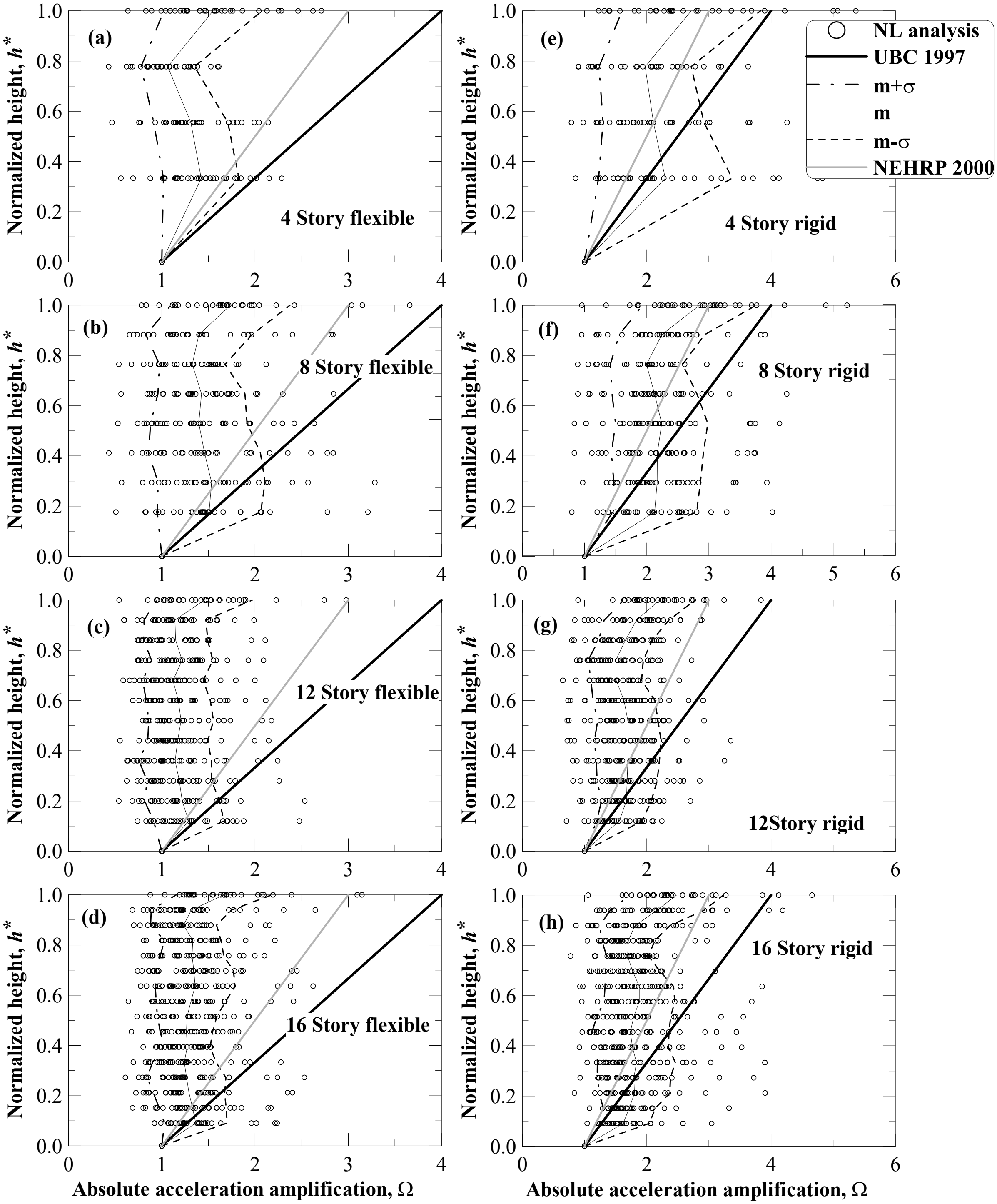
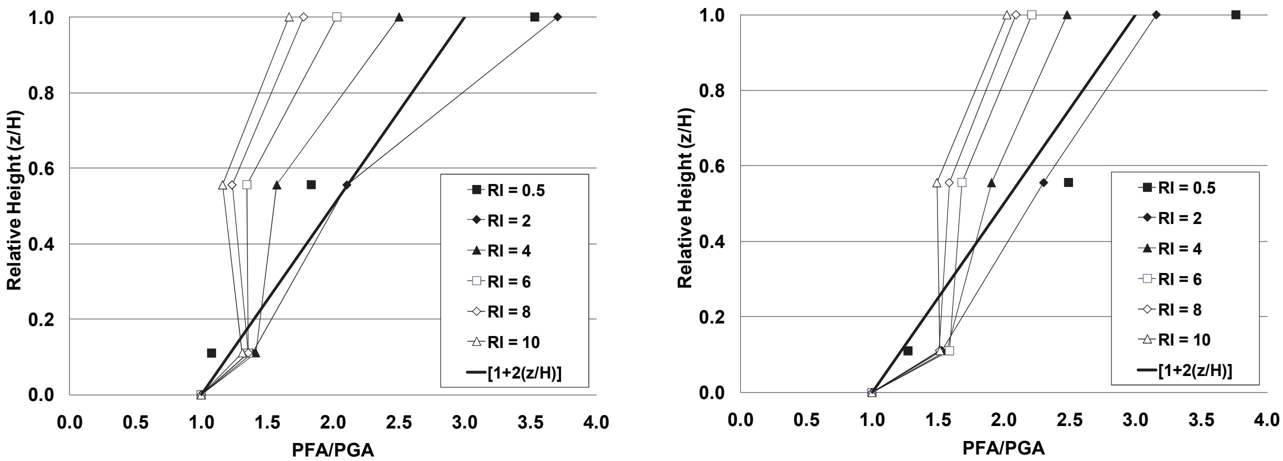


Figure 7-5 Distribution of absolute acceleration amplification for 4-, 8-, 12-, and 16-story flexible and rigid frames (Chauduri and Hutchinson, 2004).



(a) Fundamental period of 0.45 seconds (b) Fundamental period of 0.90 seconds

Figure 7-6 Normalized median peak floor acceleration (PFA/PGA) ratios for 9-story shear-wall structures exposed to increased ground motion relative intensities (Medina and Clayton, 2010).

7.3.2 Component Amplification Factor and Peak Component Accelerations

Analytical studies have incorporated the quantification of the component amplification factor for specific building structures via the estimation of the ratio PCA/PFA, which is represented in ASCE/SEI 7-16 by the parameter a_p. This ratio is period (or frequency) dependent. In the context of the estimation of F_p for flexible components, the most salient observations from these studies are presented below:

- Given an input ground motion and dynamic response characteristics of a specific structure, the magnitude of a_p is significantly influenced by: (1) the ratio of the period of the component, T_C, to the i^{th} modal period of the supporting structure, T_i; (2) the location of the component in the supporting structure; (3) the type of lateral load resisting system; (4) the damping ratios of the component and the supporting structure; (5) the level of inelastic behavior in the supporting structure and/or the component; and (6) the dynamic interaction between the component and the supporting structure.
- For elastic structures, a_p values greater than unity are obtained when the period of the component is in the neighborhood of a modal period of the supporting structure (i.e., T_C / T_i is close to one). As shown in Figure 7-7 and Figure 7-8, values of a_p greater than five have been consistently reported in many studies that have assumed a damping ratio of 5% for components and their supporting structures (Medina et al., 2006; Miranda and Taghavi, 2009; Fathali and Lizundia, 2012; Wieser et al., 2012; Sullivan et al., 2013. Values of a_p greater than 5 were typically reported for supporting structures with modal periods of less than two seconds, a period range representative of most acceleration-sensitive nonstructural components (i.e., one second or less). The value of a_p

recommended for use in ASCE/SEI 7-16 Eq. 13.3-1 and ASCE/SEI 7-16 Eq. 13.3-4 for flexible components is equal to 2.5 and it does not depend on the period of the component or the modal periods of the supporting structure. This value is generally conservative for components and supporting structures with 5%-damping ratios when the component period is longer than the initial fundamental period of the supporting structure (Medina et al., 2006; Fathali and Lizundia, 2012). Alternative approaches presented in ASCE/SEI 7-16 allow the direct calculation of a_p given the knowledge of the dynamic response characteristics of the supporting structure; however, the upper and lower limits of ASCE/SEI 7-16 Eq. 13.3-2 and ASCE/SEI 7-16 Eq. 13.3-3 still apply to the estimation of the seismic design force, F_p, which indirectly has an effect on a_p.

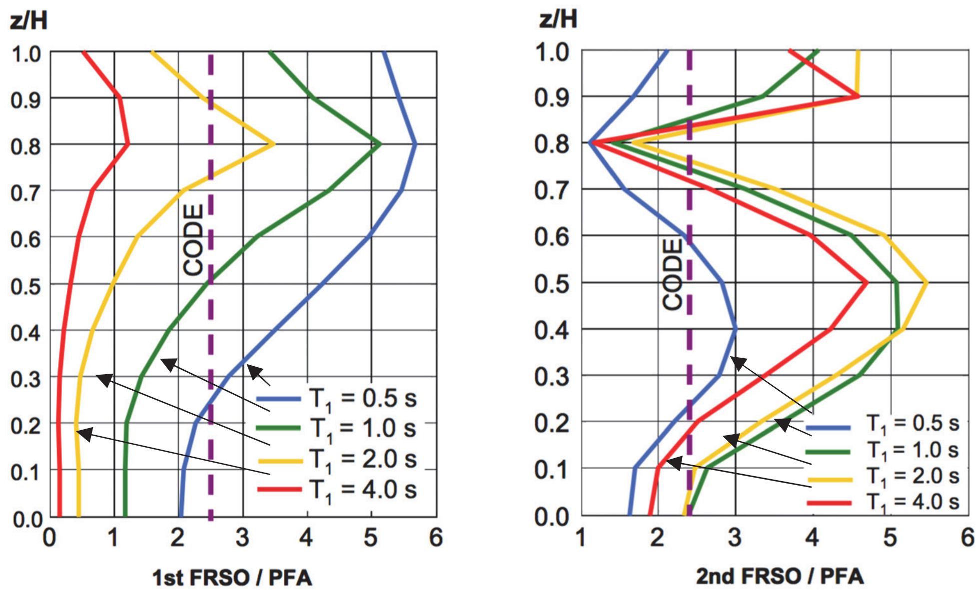


Figure 7-7 Influence of fundamental period on floor spectral ordinates of systems tuned to the fundamental and second periods of vibration of the building (Miranda and Taghavi, 2009).

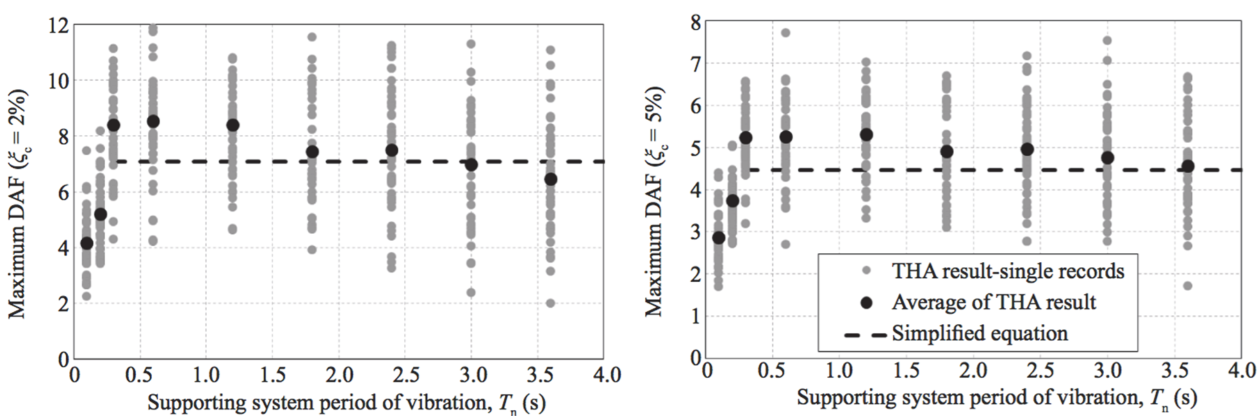


Figure 7-8 Maximum dynamic amplification factors (DAF) from linear time history analyses (THA) (Sullivan et al, 2013).

- The magnitude of the component amplification factor is strongly dependent upon the location of the nonstructural component in the supporting structure. For instance, amplifications near the top of the structure tend to be controlled by tuning of the component period with the fundamental period of the supporting structure as shown in Figure 7-7 and Figure 7-9. However, near the bottom of the structure and except for short-period (less than 0.5 second) structures, the contribution of higher modes to the magnitude of component amplifications becomes more significant (Medina et al., 2006; Miranda and Taghavi, 2009; Wang et al., 2014). The value of a_p recommended for use in ASCE/SEI 7-16 Eq. 13.3-1 and ASCE/SEI 7-16 Eq. 13.3-4 of for flexible components is constant and equal to 2.5 regardless of the location of the component in the structure. Thus, the difference between a_p values obtained from analytical studies and 2.5 is a function of location and the ratios of period of the component to the modal periods of the supporting structure. Alternative approaches presented in ASCE/SEI 7-16 allow the direct calculation of a_p; however, the upper and lower limits of ASCE/SEI 7-16 Eq. 13.3-2 and ASCE/SEI 7-16 Eq. 13.3-3 still apply to estimates of the seismic design force, F_p. The implementation of such limits indirectly provides a variation of the estimated force demand value with height.

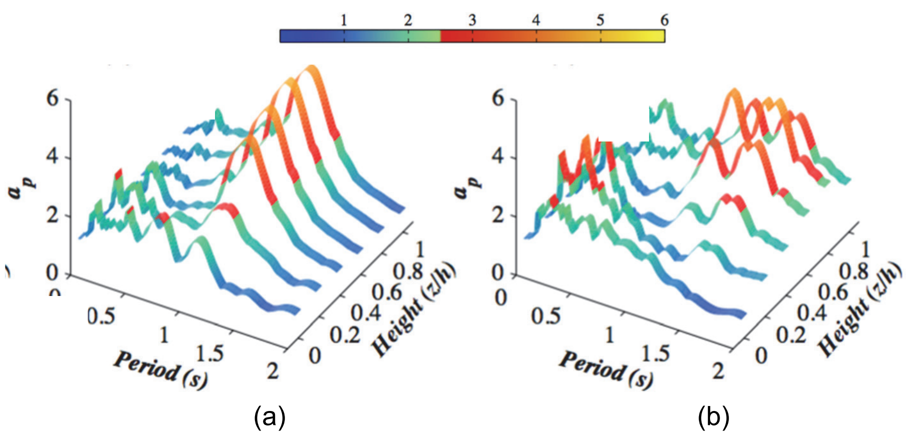


Figure 7-9 Component amplification factor for BNCS building subjected to two different ground motions with equivalent 5%-damped short-period spectral accelerations of: (a) 0.47g; and (b) 1.38g (Wang et al., 2014). (To complement the z-axis scale, a_p > 2.5 is denoted by warm colors and a_p < 2.5 is denoted by cool colors.)

- Component amplifications tend to be larger for moment-frame buildings when the component period is near the fundamental period of the supporting structure. However as shown in Figure 7-10, when the component period is close to the higher modal periods of the supporting structures, structural-wall or braced-frame systems that deform in flexure exhibit the largest component amplification factors (Taghavi and Miranda, 2006; Medina et al., 2009). The value of a_p recommended for use in ASCE/SEI 7-16 Eq. 13.3-1 and ASCE/SEI 7-16 Eq. 13.3-4 does not depend on the type of lateral-load resisting system.

Alternative approaches presented in ASCE/SEI 7-16 allow the direct calculation of a_p; however, the upper and lower limits of ASCE/SEI 7-16 Eq. 13.3-2 and ASCE/SEI 7-16 Eq. 13.3-3 still apply to estimates of the seismic design force, F_p, which indirectly has an effect on a_p.

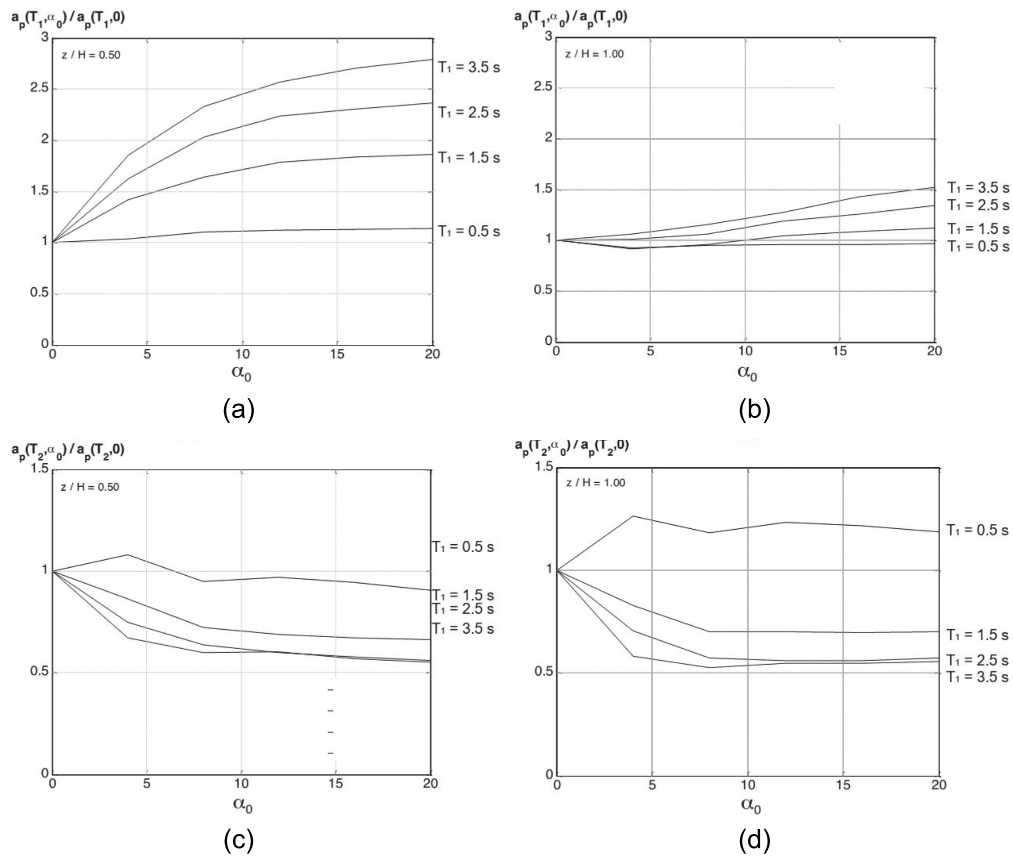
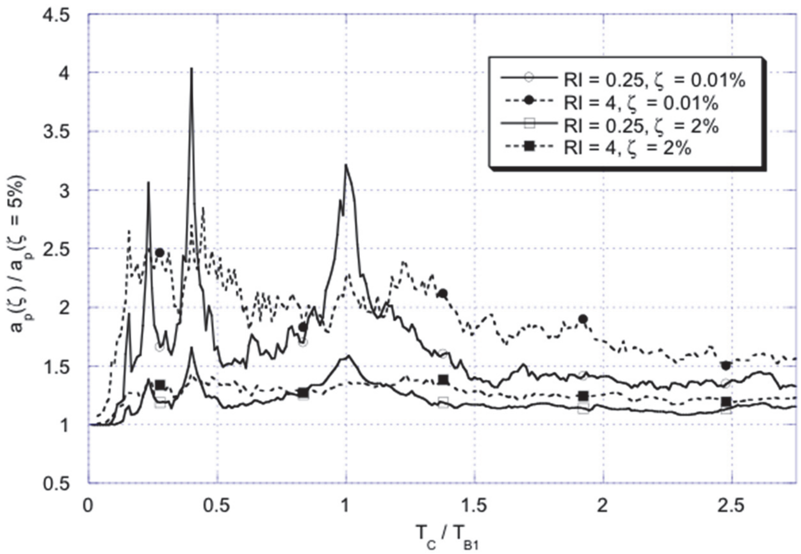


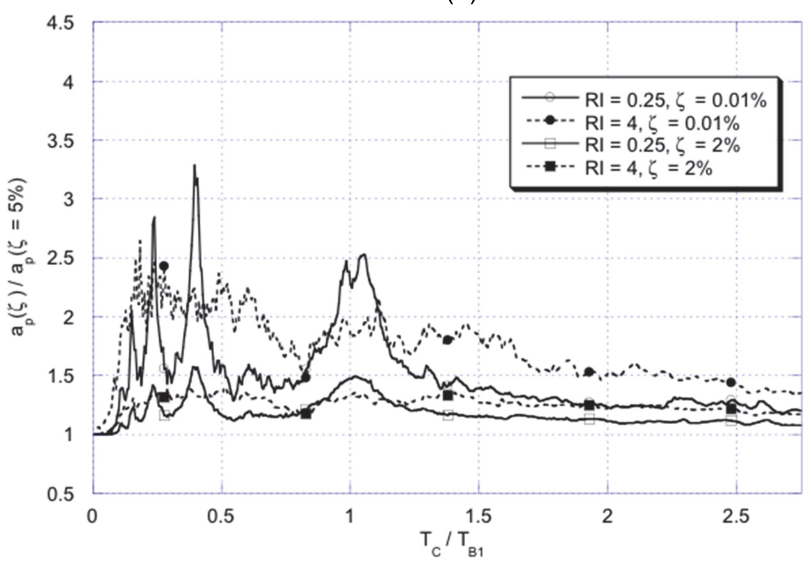
Figure 7-10 Effect of lateral stiffness ratio, α_0, on component amplification (a_p ratio for shear-type building to flexural building) considering: (a) first mode of vibration and located at mid-height; (b) first mode of vibration and located at roof; (c) second mode of vibration and located at mid-height; and (d) second mode of vibration and located at roof (Taghavi and Miranda, 2006).

- As shown in Figure 7-11 (and also in Figure 7-8), values of a_p for component damping ratios of 2% are approximately 1.5 times larger than those obtained for component damping ratios of 5% (e.g., Medina et al., 2006; Sullivan et al., 2013). Smaller component damping ratios will result in larger amplifications. The value of a_p recommended for use in ASCE/SEI 7-16 is based on a 5% component damping ratio. Alternative approaches that include the direct calculation of the component amplification factor are also based on a damping ratio of 5%.

- Inelasticity in the supporting structure influences the magnitude of the component amplification factor in two primary ways: (1) inelasticity reduces PCA demands for components with periods near the initial fundamental period of



(a)



(b)

Figure 7-11 Median component amplification factors for elastic and inelastic 9-story buildings with 0.01 and 2% damping ratios and frames considered to be: (a) rigid; and (b) flexible (Medina et al., 2006).

the supporting structure; and (2) for components with periods in tune with the higher modal periods of the supporting structure, PCA demands are reduced primarily for moment-frame structures. In general, as shown in Figure 7-12 and Figure 7-13, structural-wall (flexural-type) systems do not experience a significant decrease in PCA demands due to inelasticity in the primary structure, as compared to the reductions exhibited by moment-frame structures, when the period of the component is in tune with higher modal periods of the supporting structure. The thin gray lines in these figures correspond to a floor response spectrum for an individual design-level ground motion and the thick gray line is the 50^{th} percentile (or median).

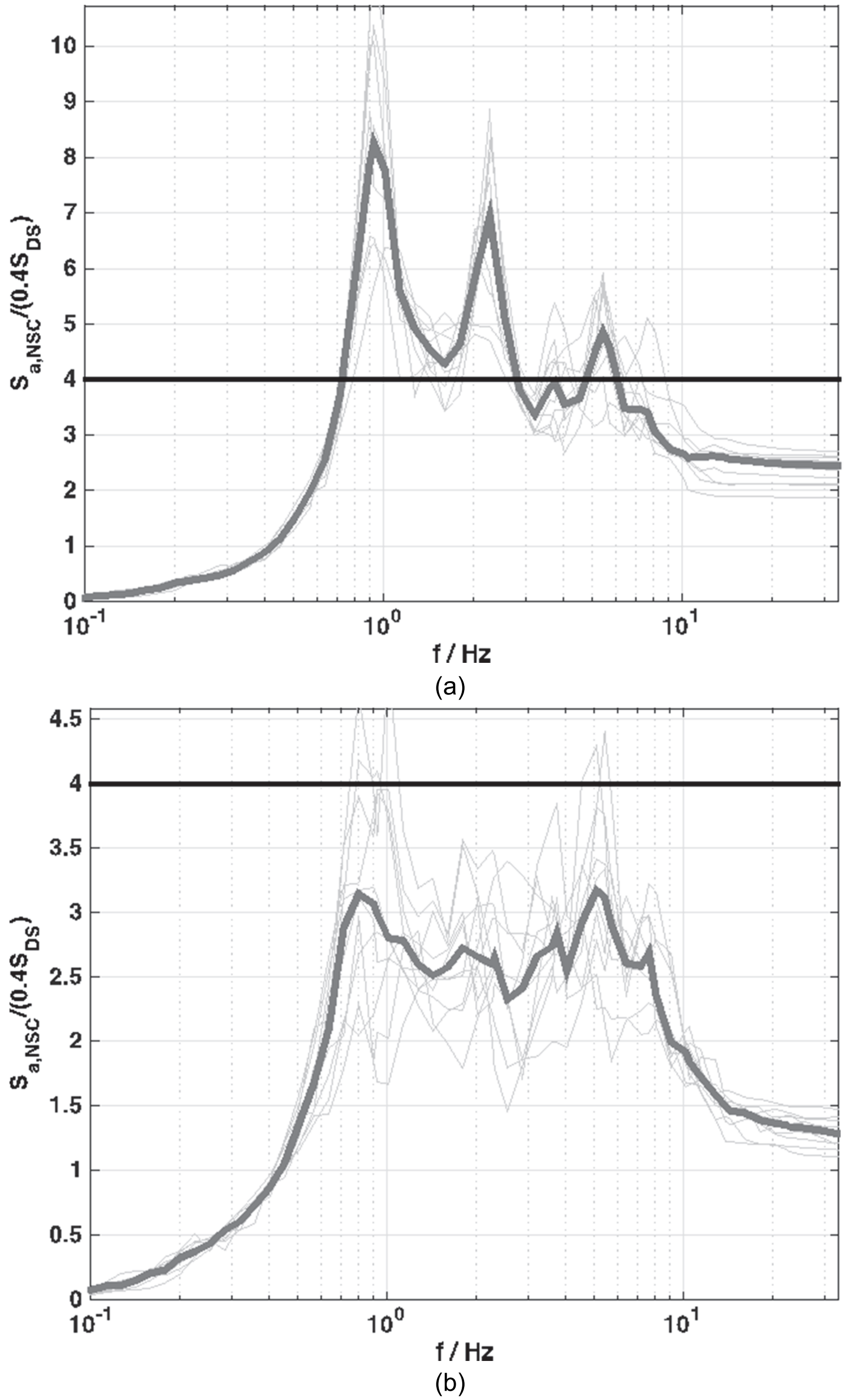


Figure 7-12 Floor response spectra from various design-level ground motions for eighth floor level of 9-story: (a) elastic frame structure; and (b) inelastic frame structure designed with a ductility-dependent response modification factor of 3.

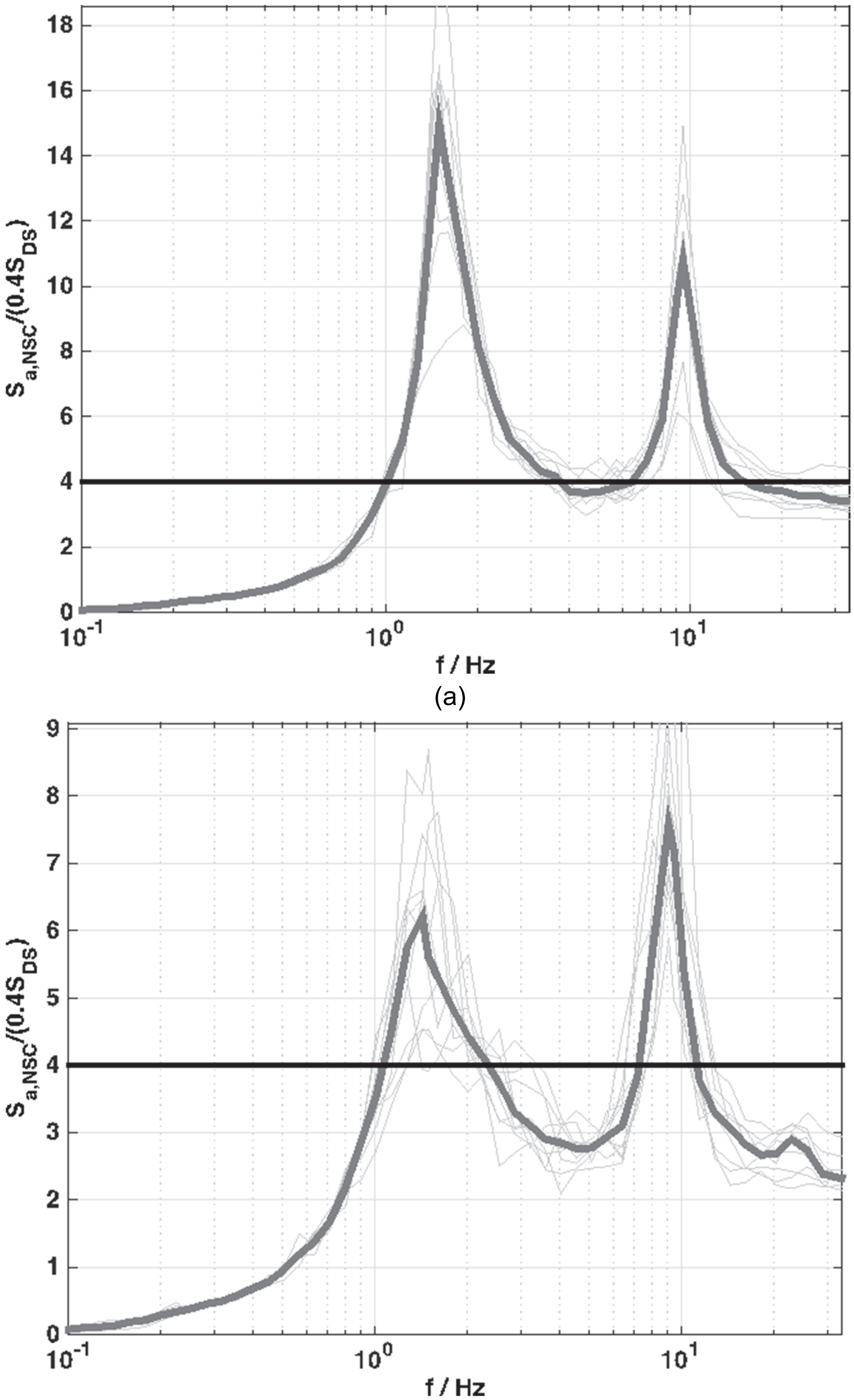


Figure 7-13 Floor response spectra from various design-level ground motions for eighth floor level of 9-story: (a) elastic structural-wall system; and (b) inelastic structural-wall system designed with a ductility-dependent response modification factor of 3.

- When certain conditions are met: (components with damping ratios smaller than 5% located near the bottom of the supporting structure with periods in the vicinity of higher modal periods of the supporting structure), inelasticity in the primary structure can increase the magnitude of component acceleration demands and component amplification factors (Sankaranarayanan and Medina, 2007; Chauduri and Villaverde, 2008; Medina et al., 2009). ASCE/SEI 7-16 Eq. 13.3-1 and ASCE/SEI 7-16 Eq. 13.3-4 utilize values of a_p that account generically and subjectively for: (1) the influence of inelasticity in the supporting structure; and (2) period shifting of the component caused by its nonlinearity via the implementation of the upper limit of ASCE/SEI 7-16 Eq. 13.3-2. Direct estimates of this parameter can be obtained via the development of floor response spectra from nonlinear response history procedures.

- As shown in Figure 7-14, dynamic interaction effects are important when the mass of the structural components is greater than approximately 1% of the mass of the supporting structure (Taghavi and Miranda, 2008). The value of a_p recommended for use in ASCE/SEI 7-16 Eq. 13.3-1 and ASCE/SEI 7-16 Eq. 13.3-4 is applicable to conditions in which dynamic interaction effects can be neglected. Interaction effects can be considered in the development of floor response spectra by conducting response history analyses that utilize coupled models.

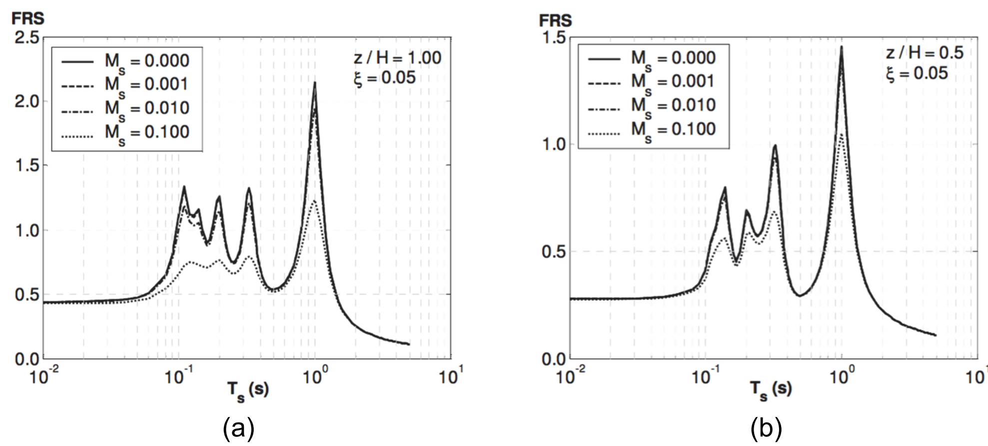


Figure 7-14 Effect of mass ratio, M_s, on floor response spectrum of a coupled primary-secondary system located at: (a) roof; and (b) mid-height (Taghavi and Miranda, 2008).

7.3.3 Upper and Lower Limits on F_p and Floor Response Spectrum Values

- Previous sections of this chapter have provided a brief overview of lessons learned from the evaluation of the in-structure amplification and the component amplification factors based on results from analytical studies. However, in order to perform a comprehensive evaluation of code-based estimates of F_p based on analytical studies, it is necessary to conduct a direct comparison between total component accelerations obtained from analysis and ASCE/SEI 7-16 estimates that incorporate the upper and lower limits defined by ASCE/SEI 7-16 Eq. 13.3-2 and ASCE/SEI 7-16 Eq. 13.3-3.

- Medina and Clayton (2010) and Medina (2013) present estimates of F_p with $I_p = 1$ and $R_p = 1$ for flexural systems exposed to design-level ground motions. These results suggest that estimates of F_p, which in many cases are controlled by the upper limit of ASCE/SEI 7-16 Eq. 13.3-2, can be smaller than those obtained based on analytical studies for flexible components located near the top of the supporting structure and with periods close to the higher modal periods of the supporting structure. For instance, the solid horizontal black lines in Figure 7-12 and Figure 7-13 represent the upper limit for F_p. It is evident from these figures that when the structures are elastic, the upper limit is significantly exceeded. For flexural systems exposed to design-level ground motions and designed for a response modification factor of three (3), the upper limit can also be exceeded for a specific structure as shown in bottom of Figure 7-13. Estimates of F_p are generally comparable between ASCE/SEI 7-16 and analytical studies for rigid components located near the top of the supporting structure and for flexible components located near the bottom of the supporting structure. However, F_p estimates for rigid components located near the bottom of the structure can be smaller than those obtained from analytical studies. Results presented in Miranda and Taghavi (2009) also show that peak component acceleration demands can be larger than those obtained via the calculation of F_p (with $I_p = 1$ and $R_p = 1$) for flexible components located in both flexural- and shear-type buildings with fundamental periods of up to 1 second. The information suggests that at the very least, a reevaluation of ASCE/SEI 7-16 Eq. 13.3-2 should be conducted to assess whether modifications to the upper limit for F_p are warranted.

7.4 Evaluation of ASCE/SEI 7-10 Equations for Seismic Design of Nonstructural Components using CSMIP Records

A study by Fathali and Lizundia (2011) was conducted to evaluate the ASCE/SEI 7-10 equations for seismic design of acceleration-sensitive building nonstructural components. The findings of that study were summarized in a paper (Fathali and Lizundia, 2012) and also provided a proposed revision to the ASCE/SEI 7-10 Eq. 13.3-1 used to calculate the design force, F_p. There were two issues that were of interest in this study. The first issue being the component amplification factor, a_p, to adjust the PFA, and the second issue being the linear transition of response based upon the height of the component compared to the height of the building.

The existing determination of a_p is based on whether the component is considered to be *rigid* (a component fundamental period of 0.06 seconds of less) or *flexible* (a component fundamental period greater than 0.06 seconds). As shown in Figure 7-15, use of an a_p spectrum is proposed, which is anchored to a plateau of $a_{p,\max}$ and reduces to unity for *rigid* components following a linear relation and also reduces for more flexible components. This spectrum allows, where appropriate, a_p magnitudes less than unity and also magnitudes between unity and $a_{p,\max}$.

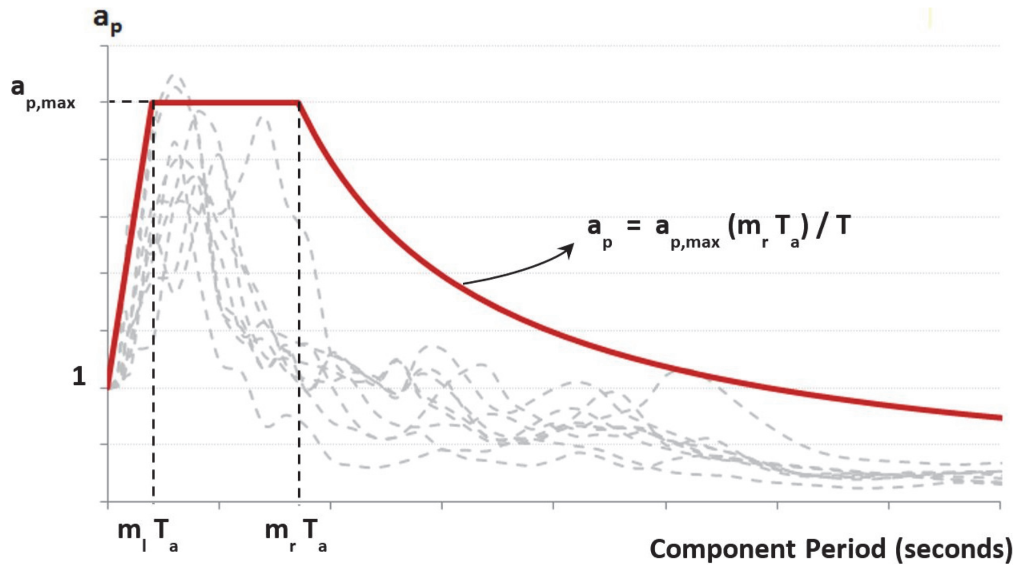


Figure 7-15 Proposed generic floor amplification factor (a_p) 5%-damping spectrum (Fathali and Lizundia, 2011).

As presented in ASCE/SEI 7-10, the transition with building height is defined the relationship $(1 + 2\ {}^{z}/_{h})$, so that the value is unity at the base of the building and three at the roof level. The new proposed equation includes two new terms, α and β. The proposed transition has the relationship of $(1 + \alpha\ ({}^{z}/_{h})^{\beta})$. The term, α, replaces the constant of 2 to provide opportunity to adjust the amplification factor, while the term, β, allows for a nonlinear transition with building height.

Using data available from the CSMIP building stations, the study considered 2,224 above ground data points to statically develop recommended values for α and β. Fathali and Lizundia concluded that these values are sensitive to both the PGA of the site and the period of the building. As a consequence, a set of tables was developed to define these parameters for evaluation of existing buildings and design of new buildings.

In general, these simple modifications to ASCE/SEI 7-10 Eq. 13.3-1 were suggested to provide a better fit to existing data from the response of buildings in actual earthquakes and yield more appropriate design or assessment forces for nonstructural components. However, damping in the building was noted as likely to be influential, but was not studied. The challenge with using the proposed equation is the need to know information about the dynamic characteristics of the building and the component in order to determine the design force.

7.5 Future Directions

Results presented in this chapter demonstrate that, as expected, response history analysis procedures can provide estimates of nonstructural component acceleration demands in a building. The implementation of such approaches, however, requires, at the very least, detailed knowledge of the modal properties of the supporting structure and the component. The reality is that, in the vast majority of cases, component properties are not known at the time that seismic design forces for supports and attachments are estimated. Even when dynamic properties are known, most estimates of F_p rely upon simplified approaches, and the conclusion is that emphasis should be placed on a comprehensive evaluation of simplified equations to estimate F_p that incorporate upper and lower limits set for F_p.

Chapter 8

Practice Issues

Reducing nonstructural earthquake losses requires not only technically sound code requirements, but also effective implementation during all phases of design and construction. There are many parties with responsibility for implementation, including design professionals, contractors, subcontractors, manufacturers, inspectors, and building officials. This chapter summarizes current practice and identifies challenges related to design and construction of nonstructural components.

In 2009, practice issues were explored in a study undertaken by the Earthquake Engineering Research Institute (EERI). This was the first effort designed to build on anecdotal evidence and provide an understanding of the reasons for compliance and noncompliance with code requirements related to nonstructural design. This study has been included in this report to provide a foundational understanding of the state of practice, create an awareness of the factors at play, and present ideas for making improvements.

In 2015, the ATC-120 *Workshop on the Application of Seismic Code Provisions and Performance of Nonstructural Components and Systems* was conducted to obtain broad-based industry input on the design, procurement, and construction of nonstructural components and systems. The workshop was intended to provide an independent data point for confirmation of information gained as a result of the background knowledge study, and to guide planning for future phases of work. It was also intended to provide context, grounding, and assistance in the prioritization of problem-focused studies and research recommended in Chapter 9.

Both the EERI study and the ATC-120 Workshop demonstrated that those involved in the design, manufacture, and installation of nonstructural components and systems agree that enforcement of related code requirements is inconsistent and, in some cases, nonexistent. Many believe that enhanced plan review and construction inspection of nonstructural components and systems would have the greatest impact on improving nonstructural performance in earthquakes and reducing future nonstructural losses. Workshop participants also identified specific design issues for which additional guidance is needed.

8.1 EERI Study

EERI undertook an in-depth study entitled, *Identification of Methods to Achieve Successful Implementation of Nonstructural and Equipment Seismic Restraints*

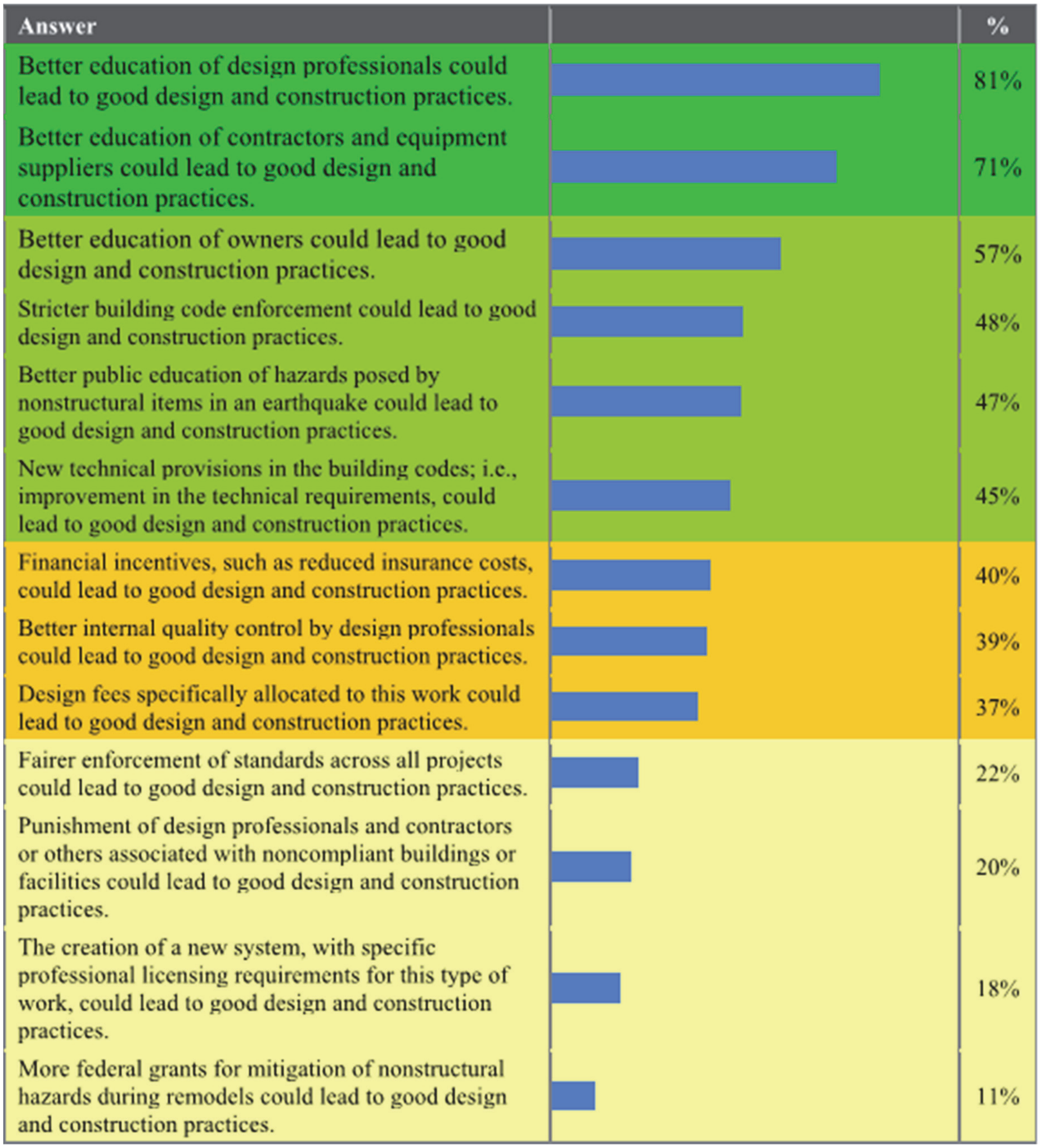
(Masek and Ridge, 2009), to help achieve nonstructural seismic protection as matter of standard practice. The EERI study focused on identifying the primary inhibiting and enabling factors affecting nonstructural seismic design and construction practices. Factors included perceptions of current compliance with existing codes and standards, why compliance was lower than required by building codes, and who should be responsible for nonstructural seismic design and construction. Interdisciplinary communication gaps and resulting procedural problems among the owner, design team, code enforcement groups and the construction team were also explored. Data was collected over a period of twelve months from telephone interviews, in-person interviews, and an online technical survey tool. These data-gathering efforts were done across the common disciplines involved in construction projects: owners, designers, code enforcement professionals, equipment suppliers, and contractors. A total of 301 practicing professionals contributed to the study. Most participants (86%) were from California, Washington, and Utah.

There was consistent agreement that the state of practice for nonstructural seismic design and construction was not adequate, with agreement that noncompliance with current building codes occurred frequently. Using data gathered from the target respondent groups, the authors identified 34 primary causative and corrective factors, which were organized into four categories: (1) educational factors; (2) factors related to definition of design responsibility; (3) factors related to plan review and building inspection processes; and (4) economic factors relating to design fees and construction costs. Survey respondents consistently emphasized the need for increased education across all groups (owners, designers, contractors, code enforcement professionals, and others), as well as the need to clearly define who is responsible for nonstructural seismic design. Stricter code enforcement was also recommended.

The reasons identified for lack of compliance with nonstructural requirements, and recommendations for improved implementation of seismic design requirements are summarized in Figure 8-1 and Figure 8-2. These results are aggregated across all disciplines surveyed. The percentage of respondents identifying the factor as a reason for compliance or noncompliance was used to rank the responses. Since each respondent was able to identify multiple factors, the percentages within the tables do not sum to one hundred percent. The color coding provides arbitrary groupings of the responses based upon the percentages obtained.

Answer	%
No one is adequately trained to make sure the standards are complied with.	44%
There is little regulatory enforcement of compliance with the standards.	42%
No one knows who is ultimately responsible for compliance.	40%
There is a communication breakdown between everyone involved in constructing the building that contributes to noncompliance.	35%
It is too expensive to comply with the standards.	35%
Compliance just falls through the cracks.	33%
Everyone just assumes someone else will make sure that compliance occurs.	31%
Everyone passes the buck to someone else, so compliance doesn't occur.	26%
The standards are too difficult to understand.	25%
There is little incentive to comply with the standards.	23%
Penalties for noncompliance aren't severe enough to make compliance a high priority.	20%
It is too time consuming to comply with these standards.	17%
There is a lack of compensation for oversight compliance.	15%
The likelihood of a damaging earthquake in this area is too low to justify complying with the standards.	13%
The standards are poorly designed.	12%
No one is really interested in making sure the standards are complied with.	11%
It's not against the law to fail to comply with the standards; compliance will happen when it's the law.	11%
No one is qualified to certify compliance.	9%
The likely consequences of an earthquake in my area are too minimal to justify the cost and effort of compliance.	8%
I have never seen compliance.	3%
There are liability concerns that prevent compliance.	2%

Figure 8-1 Reasons for noncompliance and their relative importance (Masek and Ridge, 2009).



Answer	%
Better education of design professionals could lead to good design and construction practices.	81%
Better education of contractors and equipment suppliers could lead to good design and construction practices.	71%
Better education of owners could lead to good design and construction practices.	57%
Stricter building code enforcement could lead to good design and construction practices.	48%
Better public education of hazards posed by nonstructural items in an earthquake could lead to good design and construction practices.	47%
New technical provisions in the building codes; i.e., improvement in the technical requirements, could lead to good design and construction practices.	45%
Financial incentives, such as reduced insurance costs, could lead to good design and construction practices.	40%
Better internal quality control by design professionals could lead to good design and construction practices.	39%
Design fees specifically allocated to this work could lead to good design and construction practices.	37%
Fairer enforcement of standards across all projects could lead to good design and construction practices.	22%
Punishment of design professionals and contractors or others associated with noncompliant buildings or facilities could lead to good design and construction practices.	20%
The creation of a new system, with specific professional licensing requirements for this type of work, could lead to good design and construction practices.	18%
More federal grants for mitigation of nonstructural hazards during remodels could lead to good design and construction practices.	11%

Figure 8-2 Recommendations for successful implementation of nonstructural seismic design provisions (Masek and Ridge, 2009).

8.2 ATC-120 Workshop

The ATC-120 *Workshop on the Application of Seismic Code Provisions and Performance of Nonstructural Components and Systems* was held on August 6, 2015 at the Center for Integrated Facility Engineering (CIFE) on the Stanford University campus. The workshop was interactive in nature and facilitated by John Kunz, an affiliate of CIFE.

Workshop participation was broad-based, consisting of invited industry experts involved in the design, manufacture, installation, and code enforcement of nonstructural components and systems. Participants were selected based on their knowledge and expertise in nonstructural component and system design and research, from geographically diverse locations, and from different size firms and different types of practices. The names and affiliations of workshop participants are provided in the list of Project Participants at the end of this report.

The workshop was intended to accomplish multiple objectives. It was intended to: (1) obtain broad-based industry input on the challenges associated with design, procurement, and construction of nonstructural components and systems; (2) provide an independent data point for confirmation of information gained as a result of the background knowledge study; (3) initiate a conversation on nonstructural performance expectations and provide information on which to base future ideas for consensus performance objectives; and (4) to provide context for eventual recommendations and guide planning for future phases of work.

Workshop discussions were organized around three sessions, each of which included plenary and breakout group activities. The sessions were structured to address the following topics:

1. Assessment of Existing Code Requirements, in which participants evaluated the effectiveness of current code requirements and identified needed improvements;
2. Nonstructural Performance Goals, in which participants were asked to judge acceptable levels of nonstructural damage under various scenarios to establish consensus performance expectations; and
3. Problem-Focused Opportunities for Change, in which participants identified the most important topics for advancing nonstructural design and construction practice.

8.2.1 Assessment of Existing Code Requirements

In this session, workshop participants were asked to evaluate nonstructural component design and analysis requirements contained in ASCE/SEI 7-10 including Supplement No. 1, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2013), and to identify aspects that they believe work well, and aspects that should be changed to improve nonstructural performance in future earthquakes.

The following aspects of existing code requirements were identified as working well:

- Hospitals designed under OSHPD jurisdiction have a high probability of providing continued service after an earthquake. Rigorous plan review and construction oversight were identified as contributing to the expected good performance.

- Building code requirements for life-safety are generally adequate. There was a general belief that the current code provisions are adequate to limit most serious injuries and avoid casualties. Force levels were generally judged to be reasonable and sufficient.
- Given that it is essential for manufacturers of nonstructural components to have a generic basis for seismic qualification of products, the criteria provided ASCE/SEI 7-10 is adequate.

The following aspects of existing code requirements were identified as needing improvement:

- Enforcement of nonstructural code provisions is not uniform among jurisdictions having authority, and varies widely across the country.
- Field quality control related to nonstructural components is lacking in most jurisdictions (with some notable exceptions such as in California hospitals and schools). Design professionals of record do not generally perform comprehensive inspections of nonstructural component installations, and are generally not paid to perform compliance inspections.
- Nonstructural code compliance centers on components. Individual components may perform well, however systems may not function, and essential services may be compromised as a result.
- The code does not have jurisdiction over stability of contents and furniture, which may pose seismic safety risks. It may be appropriate to expand code provisions to include components weighing under 400 pounds with a high aspect ratio that can overturn, impact occupants, or block egress routes or exits.
- Differential movement and drift requirements for nonstructural components are not well understood and need more clarification.
- The code does not include explicit guidance on which components are required for post-earthquake operability of an essential facility. Consequently, there is not uniform compliance with related code provisions.
- There is concern that the upper limit of F_p, provided by ASCE/SEI 7-10 Eq. 13.3-2, should not apply when modal analysis procedure is used to determine F_p in conjunction with ASCE/SEI 7-10 Eq. 13.3-4.
- It is not clear that post-earthquake functionality is ensured when components are designed with an R_p greater than unity.
- Code performance objectives are unclear and should be matched with ASCE/SEI 41-13 language, especially for displacement-governed equipment.

- Definitions in ACSE/SEI 7-10 of “High,” “Limited,” and “Low” deformability elements and attachments are needed. Examples would be helpful.
- Loads required for concrete anchor design are believed to be too high, and result in larger than needed anchors or the inability to use certain types of anchors. Reexamination of the omega factor is recommended.
- Architects and mechanical, electrical, and plumbing design professionals do not generally understand the nonstructural component design provisions in ASCE/SEI 7-10. This standard should be more clearly written, or contain introductory language in the commentary, to help architects and MEP engineers know which of their elements require design and seismic restraint.
- The response of nonstructural components and systems to vertical ground motion needs further investigation, particularly for performance of buildings in Risk Category III (a class of buildings which could pose a substantial risk to human life if failure occurred, but not considered to be essential) and Risk Category IV (a class of buildings considered to be essential and failure could pose a substantial hazard to a community).
- Seismic qualification of “designated seismic systems” is believed to be too stringent in some situations and should be revisited.

8.2.2 Nonstructural Performance Goals

In this session, workshop participants were separated into four groups, and were asked to examine 10 nonstructural damage scenarios and judge whether the observed level of damage was acceptable (or not). The scenarios considered buildings that were assigned to Risk Category II (a class of ordinary buildings with typical residential, office, or commercial use) or Rick Category IV, and considered three different levels of earthquake shaking intensity: moderate, Design Earthquake (DE), and Maximum Considered Earthquake (MCE). The photographs and descriptions used to present each nonstructural damage scenario are provided in Appendix A.

This exercise was intended to identify if a consensus on acceptable performance exists, learn what might be considered acceptable, and to provide information on which to base future ideas for consensus performance objectives. Results are summarized in Table 8-1. The entry in each cell is based on votes from each of the four teams. If all four teams agreed that the damage is acceptable, the cell is identified as “Acceptable”. If a majority (three out of four teams) agreed that the damage is acceptable, the cell is identified as “Acceptable*”. If the vote was evenly split, the cell is identified as “No Consensus” and is highlighted in the table. If a majority (three out of four teams) agreed that the damage is not acceptable, the cell is identified as “Not Acceptable*”. If all four teams agreed that the damage is not acceptable, the cell is identified as “Not Acceptable”.

Table 8-1 Summary of Responses for Acceptable Levels of Nonstructural Damage

Situation of Interest		Seismic Hazard	Performance Assessment	
#	*Description*		*Risk Category II*	*Risk Category IV*
1	Lightweight exterior cladding is damaged and requires replacement. No safety concern.	Moderate	Not Acceptable*	Not Acceptable
		Design	Acceptable	Not Acceptable
		MCE	Acceptable	Acceptable
2a	Veneer pieces weighing 20 pounds or less fall, but not above a building egress.	Moderate	Not Acceptable	Not Acceptable
		Design	No Consensus	Not Acceptable
		MCE	Acceptable*	No Consensus
2b	Veneer pieces weighing 20 pounds or less fall, and above a building egress.	Moderate	Not Acceptable	Not Acceptable
		Design	Not Acceptable	Not Acceptable
		MCE	No Consensus	Not Acceptable*
2c	Veneer pieces weighing more than 20 pounds fall.	Moderate	Not Acceptable	Not Acceptable
		Design	Not Acceptable	Not Acceptable
		MCE	No Consensus	Not Acceptable
3	Gypsum wallboard cracks.	Moderate	Acceptable	No Consensus
		Design	Acceptable	Acceptable
		MCE	Acceptable	Acceptable
4	Suspended light fixture is dislodged.	Moderate	Not Acceptable	Not Acceptable
		Design	Not Acceptable	Not Acceptable
		MCE	Acceptable	Not Acceptable*
5	Five-foot tall furniture falls over.	Moderate	No Consensus	Not Acceptable
		Design	No Consensus	Not Acceptable
		MCE	Acceptable	Not Acceptable*
6a	A sprinkler head is damaged from contact with an adjacent component and water is released.	Moderate	Not Acceptable*	Not Acceptable
		Design	Acceptable*	Not Acceptable
		MCE	Acceptable	No Consensus
6b	Five percent of sprinkler heads are damaged and water is released.	Moderate	Not Acceptable	Not Acceptable
		Design	No Consensus	Not Acceptable
		MCE	Acceptable	Not Acceptable*
6c	Leaks or breaks in five percent of domestic cold water pipe runs.	Moderate	Not Acceptable	Not Acceptable
		Design	No Consensus	Not Acceptable
		MCE	Acceptable*	Not Acceptable*
7	Equipment anchorage fails and equipment is damaged and requires replacement or repair.	Moderate	Not Acceptable	Not Acceptable
		Design	Not Acceptable*	Not Acceptable
		MCE	Acceptable	Not Acceptable
8	Acoustic tile ceiling grid is damaged and requires replacement.	Moderate	Not Acceptable	Not Acceptable
		Design	Acceptable	Not Acceptable
		MCE	Acceptable	Not Acceptable*
9	Exit doorway is jammed and cannot be opened manually.	Moderate	Not Acceptable	Not Acceptable
		Design	Not Acceptable	Not Acceptable
		MCE	Not Acceptable*	Not Acceptable
10a	In a public facility, items each weighing 20 pounds or less fall from 4 feet in height.	Moderate	Acceptable	Not Acceptable*
		Design	Acceptable	No Consensus
		MCE	Acceptable	Acceptable
10b	In a public facility, items each weighing more than 20 pounds fall from 4 feet in height.	Moderate	Not Acceptable*	Not Acceptable*
		Design	Acceptable*	Not Acceptable*
		MCE	Acceptable*	No Consensus
10c	In a public facility, items each weighing 20 pounds or less fall from over 8 feet in height.	Moderate	Not Acceptable*	Not Acceptable
		Design	Not Acceptable*	Not Acceptable
		MCE	Acceptable*	Not Acceptable
10d	In a public facility, items each weighing more than 20 pounds fall from over 8 feet in height.	Moderate	Not Acceptable	Not Acceptable
		Design	Not Acceptable	Not Acceptable
		MCE	Not Acceptable	Not Acceptable

There were considerable differences of opinion about what constituted acceptable damage in the scenarios presented. In particular, there was no agreement as to whether there should be any nonstructural performance objectives associated with ground shaking from the Maximum Considered Earthquake (MCE) at a given site. Additional areas of disagreement included the following:

- Damage to contents in Risk Category IV buildings in Design Earthquake (DE) ground shaking.
- Performance of adhered veneer and similar components that can become dislodged and potentially become falling hazards.
- Damage to sprinkler systems.
- Damage to gypsum wallboard in moderate earthquake ground shaking for Risk Category IV buildings.

8.2.3 Problem-Focused Opportunities for Change

Over the course of the workshop, issues that arose repeatedly were identified as topics for detailed discussion in breakout groups. Each breakout group was asked to develop a list of actions to address the assigned issue of concern. Participants were allowed to move between groups, and some individuals participated in more than one group. The following four topics were addressed:

1. **Need for inspection of nonstructural components to be certain of proper selection, installation (attached to building), and connection, as appropriate, to MEP distribution systems.** There was a general consensus of concern that unless the inspection of nonstructural components and systems was required for code compliance and conformity with design documents and specifications, measurable improvements in performance would be unlikely even if design methodologies changed. In particular, the participants noted the following:
    - An inspection manual is needed for nonstructural components and systems. It was suggested that the *Special Inspection Manual* (CPD-DS, 2012) developed by the Development Services group within the City Planning and Development office in Kansas City, Missouri could be used as guidance in the development of a new special inspection manual for nonstructural elements and components. Another document suggested as a reference was the UFC 3-301-01.1, *Uniform Facilities Criteria - Structural Engineering* (DOD, 2014), that provides modification to or additional provisions to the *International Building Code* for use by the Department of Defense. Chapter 2-2 in UFC 3-301-01.1 provides limited information about the design and inspection of nonstructural components and systems.
    - The authority having jurisdiction (AHJ) must enforce inspections.

- Special Inspector of Record for nonstructural components should be defined to ensure that someone holds responsibility.
- Continuing education should be required for those who specify nonstructural designs.
- Training for installers is needed.
- Code support material should explain cost-benefit tradeoffs and potentially explain a process for performing a cost-benefit analysis.
- Code support material should emphasize the engineer's role in field observation and checking.
- Qualification and training guidelines should be created for nonstructural Special Inspectors of Record.

2. **Need for evaluation of parameters, a_p, R_p, I_p, and Ω_0 used in nonstructural component and system design, and development of a viable evaluation techniques.** There was considerable agreement that the parameters, a_p, R_p, I_p, Ω_0, and Appendix D of ACI 318-11, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2011b), needed to be reevaluated. In particular, the participants wanted:
   - Verification that Ω_0 is needed and, if so, calibration of Ω_0 with ACI 318 Appendix D.
   - Use of current nonstructural test data to tie a_p (from ICC-ES AC156 tests) and R_p (from UCSD Building Nonstructural Components and Systems and UNR Grand Challenge tests) to appropriate performance level MCE_R or some appropriate portion of MCE_R.
   - Examination of R_p and revise as needed for consistency and reasonableness. They noted the values of R_p for stairs, large pipes and pipe risers seemed too high.
3. **Need for modifications in Chapter 13 of ASCE/SEI 7-10 to better address deformation compatibility of nonstructural components and systems.** Participants were concerned that the current provisions in ASCE/SEI 7 do not adequately address deformation compatibility. They suggested the following actions:
   - Identification of the expected performance of nonstructural components and systems at imposed displacements and harmonization with the design provisions.
   - Inclusion of commentary in ASCE/SEI 7 to clarify how to combine F_p and imposed displacements.

- o Decision, with appropriate validation, on whether calculated displacements should be divided by R_p. The NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 2003 Edition, Part 1: Provisions (FEMA, 2004a) indicate it should be, but others suggest that it should not be.
- o Clarification of design requirements for pipe and duct risers.
- o Development of explicit design requirements for exit stairs.
- o Modification of existing requirements in Section 13.5.3 of ASCE/SEI 7-10 including Supplement No. 1 to permit other ductile materials.

4. **Need to determine the biggest risks associated with interactions of nonstructural systems and what can be done to address them.** Participants were concerned about damage resulting from interaction of nonstructural systems, and the interdependency of systems for maintaining functionality. In particular, participants suggested:
   - o Development of better details to accommodate relative seismic response displacements (e.g., precast to glass).
   - o Better understanding of the consequences of failure of one nonstructural system to other nonstructural systems (e.g., broken water pipe that shorts out an electrical subsystem).
   - o Reduction of constructability issues (e.g., installation using bracing that creates conflicts and impairs ability to comply with code) through more robust conflict resolution during the design phase, or more latitude in the construction phase.
   - o Integration of seismic bracing of co-located systems (e.g., in construction documents), more robust inspection requirements for nonstructural seismic bracing, and requirements for system interdependency analysis for critical systems (e.g., IT systems).
   - o Investigation of whether more education of engineers, architects, and contractors, rather than additional codification, would be more effective in improving the performance of nonstructural components and systems.

8.2.4 Workshop Recommendations

The workshop did not include an opportunity for the group to synthesize the discussions into a comprehensive set of prioritized recommendations. However, the following themes and related issues were extracted from the discussions:

Code Requirements

The design force equations contained in Chapter 13 of ASCE/SEI 7-10 are generally believed to provide reasonable results. There is a desire for refinement and validation in the following areas:

- The definitions of rigid and flexible often do not relate well to actual components. There are few components that are “rigid” by code definition. Also, clarification is needed on how to design a component that is considered to be rigid while the support is considered to be flexible.
- The parameters, a_p and R_p, individually suggest more precision than is warranted. There is a belief that these two parameters should be combined into a single parameter.
- Amplified forces used for anchor design (Ω_0) seem too high in most cases, and should be calibrated and made consistent with ACI 318, Appendix D.
- Additional guidance is needed for designers to consistently apply nonstructural deformation compatibility requirements.

Practice Issues

Improvements in the equations for the design of nonstructural components and systems may not lead to overall significant improvements in the performance of nonstructural components and systems. In order to be more effective, the following issues should be considered in parallel:

- Nonstructural performance objectives need to be clarified. Clear performance objectives are needed to assess the adequacy of code provisions for achieving the desired level of performance.
- More uniform enforcement (design, selection of components and anchors, and installation of nonstructural code requirements is needed.
- Construction inspection of nonstructural components is generally lacking. Development and adoption of inspection requirements is recommended.

Chapter 9
Recommendations for Further Study and Research

This report highlights important advancements in the field of nonstructural seismic design over the past several decades. More than 50 years ago, the vulnerabilities of nonstructural components in modern buildings were identified following the 1964 Great Alaska Earthquake, and recommendations for reducing the vulnerabilities were made. At that time, there was debate regarding seismic performance objectives for nonstructural components, and whether codes and standards should provide any level of performance other than life safety in a strong earthquake. In almost every earthquake since then, nonstructural damage has been the leading contributor to losses. The debate on nonstructural performance objectives continues. Codes and standards have evolved based on research and hard lessons learned in earthquakes. The effectiveness of the advancements in nonstructural seismic design is difficult to gauge, because, in any earthquake, only a fraction of the impacted structures have been designed using modern nonstructural design standards, and an even smaller fraction actually comply with the standards.

The design and construction of buildings is a combined effort of a large group of skilled individuals. Following the 1964 Great Alaska earthquake, the design and construction process was identified as a major impediment to good nonstructural performance. In many cases, only a few of the individuals on a project are familiar with seismic design requirements for nonstructural components. Involvement of individuals with a high level of seismic design expertise is often very limited when it comes to the architectural and MEP systems of buildings, which represent the vast majority of the investment.

Throughout this report, gaps in information and needed improvements have been identified. This chapter summarizes recommendations for further investigation, and synthesizes them into an organized plan for advancing nonstructural design in areas that will have the largest impact to public safety and economic welfare. The recommendations extend beyond code development, and include implementation and practice issues believed to be essential for making measurable progress in improving nonstructural performance in earthquakes. Technical subjects suited for NIST-sponsored research and development are specifically highlighted, and include the suggested scope and approach for proposed problem-focused studies.

9.1 Research Plan Objectives

The primary objective of this research plan is to present a list of recommended problem-focused studies that will eventually lead to the improved seismic performance of nonstructural components. This list is the result of a comprehensive background knowledge study that collected and summarized the body of available information on nonstructural design and performance, and identified areas of needed research. A workshop with participants representing a broad spectrum of the nonstructural design and construction industry was conducted in parallel to gather an independent perspective on the challenges associated with nonstructural code provisions, design guidance, and related implementation. Although there was considerable general agreement between issues identified in the background knowledge study and the workshop, workshop participants particularly emphasized concerns about the installation, attachment, and inspection of nonstructural components. Recommendations from the workshop are integrated into this plan.

A secondary objective of this plan is to recommend an approach for conducting each study and to facilitate planning for future phases of work. Each study description includes: (1) the importance of the proposed study; (2) a general methodology for completing each study; and (3) supporting studies that can be used to advance the objectives of each primary study, and can be independently selected based on the time and budget available in future phases of work.

9.2 Research Plan Overview

Problem-focused study and research topics have been grouped into six subject areas and two priority levels. Priority Level 1 studies represent efforts that are judged to be foundational for the further development of nonstructural seismic design requirements, or that will have immediate impact on the practice of design and installation of nonstructural components and systems. Priority Level 2 studies are important, but not judged to have as great an impact on overall improvement of the performance of nonstructural components and systems.

Each primary study has been broken down into smaller supporting studies. Some supporting studies are linked, and should be completed sequentially, while others are independent, and can be completed as time and budget permit. The recommended problem-focused studies and research topics are summarized in Table 9-1. It is envisioned that other organizations with an interest in the seismic performance of nonstructural components and systems could fund, or take responsibility to complete, the work related to individual studies or supporting studies as part of a coordinated plan to improve nonstructural design and construction practice.

Table 9-1 Recommended Problem-Focused Studies and Research

Priority Level	Problem-Focused Study	Description
Level 1	**1.0**	**Conduct Holistic Assessment of Current Code Design Approaches**
	1.1	Create Archetype Building(s)
	1.2	Define Ground Motions for Study
	1.3	Conduct Building Analyses and Develop Generic Floor Spectra
	1.4	Analytically Determine Demands on Nonstructural Components
	1.5	Evaluate Code Design Force Equations
	1.6	Evaluate Anchor Design Procedures
	1.7	Propose Nonstructural Components and Systems Code Changes
	2.0	**Develop Nonstructural Component and System Performance Objectives**
	2.1	Create a Framework for Nonstructural Performance Objectives
	2.2	Build Consensus for Nonstructural Performance Objectives
	3.0	**Improve Implementation and Enforcement of Code Requirements**
	3.1	Hold Workshops to Identify Opportunities for Improvement
	3.2	Develop Targeted Materials and Related Training
	4.0	**Clarify Requirements for Displacement Capability Design of Nonstructural Components and Systems**
	4.1	Determine Nonstructural Displacement Demands and Associated Acceptance Criteria
	4.2	Develop Enforcement and Inspection Criteria for Displacement Control
	4.3	Create Guidelines for Designers
Level 2	**5.0**	**Create a Plan for Post-Earthquake Reconnaissance and Data Collection**
	5.1	Develop Framework for Data Collection
	5.2	Create Data Collection Protocols
	6.0	**Conduct Component Testing**
	6.1	Identify Vulnerable Components for Testing
	6.2	Create Testing Protocols
	6.3	Conduct Tests

9.3 Recommended Studies

The following sections provide a description of each problem-focused study, along with an explanation of the motivation behind the recommendation and a proposed approach for implementation. This information is intended to facilitate planning for future phases of work, and can be used as a starting point for detailed planning that would be needed for execution of the work.

9.3.1 Conduct Holistic Assessment of Current Code Design Approaches

All factors contributing to the seismic performance of nonstructural components and systems should be comprehensively reviewed. Such a study should include the latest information from instrumented buildings, laboratory tests, and analytical studies, and should build on relevant past and ongoing research, and leverage those findings.

Why study this? Efforts to improve the seismic performance of nonstructural components have been underway for decades. These efforts have involved engineers of various disciplines, industry trade organizations, contractors, component and material suppliers, building officials, and legislative bodies. The efforts of these groups come together at only a few points: the building code and the job site. Most of their work is done in parallel, with little intercommunication. Information on the compatibility of their efforts is sparse, and it is unclear whether the sum of all their efforts produces the desired seismic performance. Assessments of current code requirements and methodologies, while valuable, are narrowly focused, and fail to capture the overall effectiveness of the nonstructural design process.

The effectiveness of various ASCE/SEI 7 equations for estimating seismic force demands on nonstructural components and systems, and whether the ASCE/SEI 7 requirements provide cost-effective protection of nonstructural components and systems, has been difficult to evaluate from recorded earthquake data. This evaluation has not been possible primarily because earthquakes that generated design-level (or greater) ground motions in populated areas have not occurred in regions with many structures designed using modern codes (i.e., last edition of the *Uniform Building Code* or the subsequent editions of the *International Building Code*). This problem is expected to persist in the near future because even if a populated area was exposed to a design-level earthquake, few, if any modern buildings will be instrumented. Even when instrumentation is present, nonstructural components, their supports, and attachments are generally not instrumented.

Therefore, it is important to analytically evaluate the effectiveness of nonstructural design force equations to assess the need for further development of the code provisions, propose improved design force equations for nonstructural components when needed, and to provide guidance in terms of when to implement different design options for nonstructural components and systems. An evaluation of design force equations for nonstructural components and systems in the context of all the different requirements for nonstructural components is needed. Assessing lateral force design procedures is also important because of the relationship between the code equations and seismic qualification approaches used for acceleration-sensitive nonstructural components and systems.

How should this be investigated? Study of design force equations for nonstructural components and systems should emphasize Eq. 13.3-1, Eq. 13.3-2, and Eq. 13.3-3 of

ASCE/SEI 7-16. The focus on these simplified equations is necessary because, in most cases, the dynamic properties of the component, and, in many cases, those of the supporting structure, are not known at the time of design and, therefore, they are the most commonly used equations in practice. If improvements to these equations are deemed to be desirable, this study should consider including the development of generic floor response spectra (i.e., floor response spectra that are not building-specific). Generic floor response spectra could be used to generate new simplified design force equations. The use of generic floor response spectra is compatible with the approach used in estimating the seismic base shear for structural systems, which is based on generic ground motion spectra. Because current work on the development of an ASCE/SEI standard for shake-table certification involves the development of generic floor response spectra, incorporation of spectra in ASCE/SEI 7 would facilitate a consistent basis between design force equations for nonstructural components and systems and testing of nonstructural components and systems.

An evaluation of ASCE/SEI 7-16 Eq. 13.3-4 should explicitly address the implementation of modified correlation coefficients when modal analysis procedures are used to estimate the parameter a_i (i.e., the acceleration at level i obtained from modal analysis). Correlation coefficients used to estimate maximum relative displacement demands in buildings are not the same as those used to estimate maximum absolute floor acceleration demands. The effectiveness of ASCE/SEI 7-16 Eq. 13.3-4 as it applies to components and structures, and whether it meets the original intent of providing improved design force estimates for nonstructural in longer period buildings has not been adequately investigated.

The evaluation should specifically consider the appropriateness of the upper bound design acceleration (ASCE/SEI 7-16 Eq. 13.3-2), as this equation governs the design of a large number of flexible components and systems in the upper levels of structures, and also establishes qualification testing requirements. The proposed study using generic floor response spectra would provide a means by which to investigate the limiting acceleration demands.

The specific steps required to apply a holistic approach to assessing nonstructural design force equations are described below in a series of interrelated but separate supporting studies.

Supporting Study 1.1: Create Archetype Building(s)

This study would define a family of modern building types to be evaluated that is representative of the building inventory in seismic-prone regions of the United States. This will result in a focus on low- and mid-rise structures. Building types that reflect the acceleration and displacement response of a significant portion of the commercial building stock should be selected. Building types should also be selected to correspond with instrumented buildings so that recorded earthquake response can be used to benchmark analytical models. As a minimum, the following seismic force-resisting systems should be included: steel and reinforced-concrete special moment resisting frames, reinforced-concrete ordinary moment resisting frames, reinforced-concrete shear walls, steel concentrically braced frame systems, and stiff wall-flexible diaphragm buildings. Where possible, buildings should be selected from recent and ongoing research projects.

Analytical models of selected building types and seismic force-resisting systems will be developed. Analytical models of buildings used in past projects and research studies should be incorporated, where available.

Supporting Study 1.2: Define Ground Motions for Study

Consistent with proposed nonstructural performance objectives (Section 9.3.2), this study will define the ground motion intensity level(s) at which performance assessments are to be conducted. Ground motion acceleration time histories should be selected with pseudo-acceleration spectra that are compatible with a target design spectrum. Where possible, ground motion characterizations from past projects and research studies should be used.

Supporting Study 1.3: Conduct Building Analyses and Develop Generic Floor Spectra

This study will conduct response history analyses to provide a benchmark for the evaluation of force demands in components, supports, and attachments. Generic floor response spectra that generally represent the responses of all buildings will be developed. These spectra could vary with height, and would be dependent on the ground input spectra to the building. They could also vary with building type and importance. However, it is recognized that fewer variables are better for use in engineering practice. Based on the findings, the feasibility of a generic floor response spectrum approach will be verified.

Supporting Study 1.4: Analytically Determine Demands on Nonstructural Components

This study will identify types of nonstructural components and systems that are representative of those used in the selected building types. Analytical models of nonstructural components and systems that include explicit modeling of supports and attachments will be developed and associated demands on components, supports and attachments, including anchors will be determined. Nonstructural component models should be calibrated with results from shake table testing reported in Chapter 6.

Supporting Study 1.5: Evaluate Code Design Force Equations

Given generic floor response spectra and associated nonstructural demands, the effectiveness of the various design force equations for nonstructural components and systems in ASCE/SEI 7-16 will be evaluated, and potential improvements identified. If improvements are needed to Eq. 13.3-1, Eq. 13.3-2, and Eq. 13.3-3 of ASCE/SEI 7-16, new design force equations for nonstructural components and systems will be developed, possibly derived from generic floor spectra. If improvements are needed to ASCE/SEI 7-16 Eq. 13.3-4, equations to calculate modal correlation coefficients to estimate absolute floor accelerations, and ultimately, maximum component accelerations will be incorporated. Guidance for implementing different design options for nonstructural components and systems will be provided. Studies to assess the appropriateness of code exemptions will be conducted, and related recommendations will be provided.

Supporting Study 1.6: Evaluate Anchor Design Procedures

Supporting Studies 1.1 to 1.5 will yield detailed information on expected anchor design demands under a range of ground motions and a broad spectrum of building types. Based on this information, the sensitivity of anchor demands on building and component/system parameters will be investigated. Additional anchor demand data will be gathered from seismic qualification testing and other past and ongoing shake table testing, where available. Potential improvements will be developed for current anchor design procedures, including recommendations on the appropriate factor of safety for post-installed anchors in concrete and masonry. This effort should be coordinated with ongoing anchor research, ACI committee activities, and the ASCE/SEI committees tasked with developing the shake table certification standard and updating the ASCE/SEI 7 standard.

Supporting Study 1.7: Propose Nonstructural Components and Systems Code Changes

Based on results of supporting studies listed above, changes to ASCE/SEI 7 design equations and procedures will be recommended (if needed) to improve the performance of nonstructural components and systems.

9.3.2 Develop Nonstructural Component and System Performance Objectives

Although there is general agreement about the importance of protecting nonstructural components from life-threatening damage and falling hazards in earthquakes, there is no consensus about what constitutes acceptable performance, particularly in ordinary buildings. Minimum code requirements are intended to protect life safety, but should also offer protection from property losses in some levels of earthquake shaking.

Explicit performance objectives for nonstructural components and systems should be developed to provide a transparent basis for code requirements.

Why study this? Future advances in design procedures for nonstructural components and systems require explicit definition of the performance objectives that underlie them. Without clarity in the objectives, nonstructural designs will not be consistent, and building owners, tenants, and designers may not understand the intended level of performance. ASCE/SEI 41 contains component-specific nonstructural performance goals, which clarify the desired performance objectives. However, ASCE/SEI 7 provides only general building performance expectations. Thus code compliance for new buildings can vary widely based on interpretation. For example, how much cladding damage is acceptable in a Design Earthquake? Does it need to prevent water infiltration? Is it acceptable for glazing to crack or pieces of cladding to become dislodged? Should there be any nonstructural performance requirements for ground shaking associated with the Maximum Considered Earthquake? The answers to these questions can have a significant impact on design and construction.

How should this be investigated? Development of nonstructural performance objectives will require building consensus across a broad group of stakeholders. The process should begin with a small team of experts tasked with creating a framework for defining performance objectives. Stakeholders should then be brought together to explore options, express perspectives and find common ground. Representation from code and standard development organizations will be essential to the process.

Supporting Study 2.1: Create a Framework for Nonstructural Performance Objectives

A framework for nonstructural performance objectives is needed to focus development activities and serve as the underlying foundation for the holistic assessment described in Section 9.3.1. A small team of nonstructural experts (4 to 6) should be tasked with developing the framework. Existing nonstructural performance characterizations (commentaries in ISO 13033, ASCE/SEI 41, and ASCE/SEI 7) should be considered during the development phase. The team should explicitly consider whether the objectives should be cast in a building-specific framework or organized on a component basis. The framework should include:

- multiple levels of ground motion (with clear definitions and the ability to communicate them in simple terms);
- all Seismic Design Categories; and
- explicit consideration of code exemptions.

Once the draft framework is established, use FEMA P-58, *Seismic Performance Assessment of Buildings, Volume 1 – Methodology* (FEMA, 2012a) to estimate expected nonstructural losses for current code compliant buildings. This will provide a platform for making adjustments and identifying potential areas of needed change in the framework.

Supporting Study 2.2: Build Consensus for Nonstructural Performance Objectives

After the investigation team has developed a plausible framework, hold consensus building workshops to critically review the proposed framework and develop performance objectives for nonstructural components and systems. Stakeholders who have participated in the development of FEMA P-58 and earthquake safety implementation programs such as the San Francisco Community Action Plan for Seismic Safety (CAPSS) should be invited to the workshop(s). Participants should also include individuals involved in writing codes and standards such as ASCE/SEI 41, ASCE/SEI 7, ACI 318, ACI 530/530.1, and ISO13033. Following the workshop(s), incorporate stakeholder feedback into a modified framework and set of performance objectives.

Disseminate the performance objectives to a broad community so that they can be further used in the development of design methodologies and assessment criteria. This will include outreach to codes and standards and professional organizations.

9.3.3 Improve Implementation and Enforcement of Code Requirements

The significance of nonstructural performance on total earthquake losses is well known, and seismic provisions for nonstructural components have been in building codes for decades, but compliance with the nonstructural requirements is inconsistent, even in regions of high seismic risk.

Why study this? Without proper implementation and enforcement, the most advanced seismic requirements for nonstructural components will be ineffective. The bulk of the nonstructural design requirements were developed and written by structural engineers, and are found in the structural engineering standard, but the majority of nonstructural components are designed, installed, and inspected by individuals without a structural engineering background or training. To improve seismic performance of nonstructural components, information on proper design, installation, and inspection must be made accessible to the individuals that actually perform the work, in a form that is practical and accessible.

How should this be investigated? Improved implementation and enforcement of code requirements will require collaboration with groups responsible for the design, installation, and inspection of seismic aspects of nonstructural components and systems, and their affiliated trade or industry groups such as the International Code Council (ICC) for building officials and inspectors, American Institute of Architects (AIA) for designers, National Fire Protection Association (NFPA) for fire sprinkler contractors, Association of General Contractors (AGC), and other trade groups for mechanical, electrical, curtain wall, and ceiling contractors. Through outreach with groups and individuals, tools, procedures or other vehicles can be identified to facilitate improved code compliance. Those vehicles with the greatest potential for positive change would be developed.

Supporting Study 3.1: Hold Workshops to Identify Opportunities for Improvement

This study will consist of a workshop with representatives of professional and trade groups to explore barriers for proper implementation of nonstructural seismic standards, and to identify concepts for improved education and outreach.

Supporting Study 3.2: Develop Targeted Materials and Related Training

Based on information obtained from the workshop, this study will develop targeted materials and related training to improve awareness of nonstructural seismic requirements, identify when seismic provisions are applicable, improve inspection of seismic bracing, and provide resources for designers. Targeted materials could include:

- Technical Briefs for mechanical and electrical engineers on seismic design of nonstructural components;
- aides to assist building officials in identifying nonstructural components requiring seismic design and detailing;
- guides on inspection of nonstructural components for seismic compliance; and
- articles in trade publications discussing the importance of seismic bracing and resources for improving design and installation techniques.

9.3.4 Clarify Requirements for Displacement Capability Design of Nonstructural Components and Systems

Displacement controlled nonstructural components such as partitions, cladding, and glazing systems make up a large percentage of the value of a building. Unlike acceleration-controlled components, where the standards provide force-based criteria for determining the adequacy of components, the criteria for displacement-controlled components is much more ambiguous. For example, Section 13.3.2 of ASCE/SEI 7-10 states that “The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.” Guidance regarding how seismic relative displacements “shall be considered” is lacking in practice. For architectural components, Section 13.5.2 states “Architectural components that could pose a life-safety hazard shall be designed to accommodate the seismic relative displacement requirements of Section 13.3.2." With the exception of glazed curtain walls, no guidance for determining the performance required to meet the intent of the standard for deformation controlled architectural components is provided. For mechanical components, Section 13.6.5.2 states that "Component supports shall be designed to accommodate the seismic relative displacements between points of support determined in accordance with Section 13.3.2." Again, no guidance for determining the performance required to "accommodate" displacements and meet the intent of the standard is provided. Procedures for developing acceptable designs for displacement controlled components are needed.

Why study this? Substantial nonstructural damage in past earthquakes has been caused by relative displacements between floors and between components. Damage includes failure of cladding due to displacements between floors, and water damage caused by interaction between adjacent components, notably sprinkler piping. A clear request from practitioners includes the need for design guidance related to displacement compatibility. Explicit design requirements, and design practices, that prevent adverse interactions could significantly reduce related earthquake losses.

How should this be investigated? To design for displacements, the anticipated displacement demands must be known. This effort, therefore, requires mining of available building shake table test data (such as those described in Chapter 6) and earthquake data, and augmenting it with numerical simulations using models developed for the holistic investigation of nonstructural components and systems (Section 9.3.1).

In addition, key contributors to nonstructural losses should be specifically investigated. This should include consideration of various cladding and wall systems, as well as piping systems. With an understanding of the displacement tolerance of these systems, and a clear understanding of targeted performance, practical design and installation guidance could be developed to reduce future losses.

Supporting Study 4.1: Determine Nonstructural Displacement Demands and Associated Acceptance Criteria

This study will conduct analytical investigations to determine displacement demands on nonstructural components and systems. This work should build on the analytical models developed in Supporting Study 1.4. In addition, quantitative data should be collected from shake table tests and qualitative data should be collected from available earthquake reconnaissance. Based on a comprehensive understanding of displacement demands, associated acceptance criteria will be developed.

Supporting Study 4.2: Develop Enforcement and Inspection Criteria for Displacement Control

Significant displacement-related damage is related to interactions among systems and it may be impractical, if not impossible, to identify at these interactions at the time of design. However, many vulnerable conditions could be identified during construction if someone is tasked with that responsibility, is aware of the conditions that lead to damage, and is knowledgeable about the related code requirements. This study will prepare practical guidelines for inspection of nonstructural components and systems, which ideally would be combined with Supporting Study 3.2. "Rules of thumb" and other simple tools will be created that can be used to provide practical guidance to inspectors and installers.

Supporting Study 4.3: Create Guidelines for Designers

This study will prepare a set of easily-understood guidelines that distill code requirements related to displacement control into easily understandable instructions for designers. Guidelines will be by system so that they can be easily extracted by system trade groups and professional organizations. Guidelines should be consistent with the performance objectives developed in Section 9.3.2.

9.3.5 Create a Plan for Post-Earthquake Reconnaissance and Data Collection

As mentioned in Chapter 2, there is a shortage of systematic quantitative investigations of seismic performance of nonstructural components and systems. Despite the recurring lessons from recent earthquakes showing that direct and indirect losses due to nonstructural damage could be far greater than those due to structural damage, the focus of reconnaissance efforts has been on seismic performance of building structural systems. The lack of quantitative information can also be attributed to: (1) the many types of and variations between nonstructural components and systems; and (2) the majority of nonstructural damage that occurs inside of buildings, which may not be immediately accessible or is not accessible before the damage is cleaned up or repaired by the building owner.

Why study this? The advancement in seismic protection of nonstructural components and systems has suffered from a lack of an essential tool that captures the performance of nonstructural components subjected to earthquake shaking. A system that captures perishable performance data would enable documentation of the performance of components, determine if the components are properly designed, installed and maintained, and estimate the financial and operational impacts associated with the performance of nonstructural components and systems in an earthquake. Findings can lead to prioritized cost-effective practice improvements that address known deficiencies.

How should this be investigated? A series of guidelines and forms should be developed for post-earthquake reconnaissance of nonstructural components and systems that also includes examples of nonstructural components that performed well during the earthquake. In addition, recommendations should be developed for how to use and enhance existing building instrumentation programs (such as the California Strong Motion Instrumentation Program, CSMIP) to provide needed response data for validating analytical studies and qualification testing.

Post-earthquake reconnaissance efforts need to develop a large enough data set for statistical analysis. The data set should include:

- descriptions and photographs of nonstructural components, systems, and anchorage;
- the design seismic force and displacement capacity of nonstructural components;
- the as-built seismic force and displacement capacity of nonstructural components;
- the seismic forces and displacement demands during the earthquake;
- a description of the performance of nonstructural components or systems during the earthquake;
- the impact of the performance of nonstructural components or systems on the continued operation and use of the building immediately after the earthquake; and
- secondary damage, if any, from the performance of nonstructural components or systems during the earthquake.

Supporting Study 5.1: Develop Framework for Data Collection

This study will create a working group tasked with developing a robust and publically available database for the collection of earthquake response of nonstructural components. The database should leverage technology to create a useful and broadly usable platform for data collection.

Supporting Study 5.2: Create Data Collection Protocols

On a system-by-system basis, this study will develop data collection tools to ensure that information obtained after an earthquake is uniformly collected and can be used for statistical analyses.

9.3.6 Conduct Component Testing

Since damaging earthquakes occur infrequently, component testing remains one of the most effective methods for evaluating the behavior and contextualizing it with performance objectives. This is particularly true for classes of nonstructural components where their type, design, and installation practices vary widely. Many tests focus on the performance of the component itself, and important aspects of behavior (such as the anchorage forces generated during the test or potential interaction with connecting distribution systems) are not captured. Unfortunately, very few component tests have been conducted while components are interconnected (more representative of configurations observed in the field). In addition, the force demands on attachments of the component to the shake table are usually not captured during the tests.

Why study this? Although analytical investigations can contribute greatly to an understanding of nonstructural component response, and can guide code development, the procedures require calibration, which can be provided with controlled testing. Absent post-earthquake data on code-compliant installations, component testing offers the means by which we can understand how selected components perform under a range of shaking intensities, and to calibrate the tools used for evaluation and design.

How should this be investigated? Shake table tests remain a key tool for assessing the behavior of both components and systems. Essential for understanding system-level interactions will be testing an arrangement of components as they are commonly used in practice, and tracking the kinematics of the system along different load paths and at identified degrees of freedom in the system. In addition, data on the forces experienced by the attachments to the table, such as anchor bolts or vibration isolators are needed.

Depending on the type of nonstructural components, such tests may be realized with model-scale tests (i.e., not using the exact nonstructural component, but by constructing models with like geometric and mass configurations). Alternatively, using actual nonstructural components, if possible, will be highly desirable. Slow cyclic tests using an arrangement of hydraulic actuators also remain a plausible testing option for assessing the performance of systems of nonstructural components. In either testing regime, design of the test matrix (selection of the most generalized configurations), selection of the test protocol, and development of a strategic layout of instrumentation to capture system-level demands will be essential. Nonstructural systems that lack knowledge include (in no order of priority): (1) mechanical and electrical equipment and their associated interconnections to the building; (2) plumbing systems; (3) stairs with representative boundary conditions; (4) heating, ventilation, and air-conditioning (HVAC) components and their various interconnections; and (5) roof-mounted equipment and their attached piping, which are sometimes laid horizontally and subsequently drop vertically into the building. To undertake a system-level component test program, first the most vulnerable systems must be identified. Subsequently, a handful of systems must be characterized in a limited number of common configurations. Finally, the testing regime needs to be defined. Much of this last aspect will depend on resources available to undertake this effort.

Supporting Study 6.1: Identify Vulnerable Components for Testing

This study will convene a task group to consider components that are significant sources of nonstructural losses, are difficult to analytically evaluate, and would most benefit from laboratory testing. A prioritized list for needed component testing will be created.

Supporting Study 6.2: Create Testing Protocols

This study will build on widely used experimental protocols, and develop a consensus-based methodology for laboratory investigation of nonstructural components and systems. Shake table testing protocols should be developed to gather data consistent with the performance objectives developed in Section 9.3.2. A framework will be developed for consistent documentation of results that allows for public distribution and use.

Supporting Study 6.3: Conduct Tests

This study will seek sources of funding and will implement the testing of nonstructural components and systems.

Appendix A

Workshop Discussions on Acceptable Nonstructural Performance

The ATC-120 *Workshop on the Application of Seismic Code Provisions and Performance of Nonstructural Components and Systems* included a discussion on acceptable performance of nonstructural components and systems given different scenarios related to component or system type, earthquake shaking intensity, and building design level. This session was intended to establish consensus on performance goals (or find lack of consensus). Teams of workshop participants were asked to look at ten examples of nonstructural earthquake damage and state whether the damage was acceptable in a building assigned to Risk Category II (buildings with ordinary residential, office, or commercial occupancies) or Risk Category IV (essential use occupancies), considering three different levels of shaking intensity: moderate, Design Earthquake (DE), and Maximum Considered Earthquake (MCE).

Figures A-1 through A-10 provide the photographs and descriptions used to present each situation that was discussed. Judgements regarding the acceptability of damage and the level of consensus in the opinions are presented in Chapter 8, Table 8-1.

A.1 Situation No. 1 – Lightweight Exterior Cladding

Figure A-1 Lightweight exterior cladding is damaged and requires replacement, but there is no safety concern due to falling panels.

A.2 Situation No. 2 – Exterior Veneer

Situation 2a: Each piece weighs 20 pounds or less, and not above a building egress.
Situation 2b: Each piece weighs 20 pounds or less, and above a building egress.
Situation 2c: Each piece weighs more than 20 pounds off any portion of building.

Figure A-2 Veneer pieces fall from a building façade.

A.3 Situation No. 3 – Gypsum Wallboard

Figure A-3 Gypsum wallboard cracks.

A.4 Situation No. 4 – Suspended Light Fixtures

Figure A-4 Suspended light fixture is dislodged and falls.

A.5 Situation No. 5 – Furniture

Figure A-5 Unsecured five-foot tall furniture topples over.

A.6 Situation No. 6 – Fire Sprinkler or Water Supply Systems

Situation 6a: One sprinkler head is damaged and water is released.
Situation 6b: Five percent of sprinkler heads are damaged and water is released.
Situation 6c: Leaks or breaks in five percent of domestic water pipe runs.

Figure A-6 Damage to fire sprinkler or water supply systems.

A.7 Situation No. 7 – Equipment

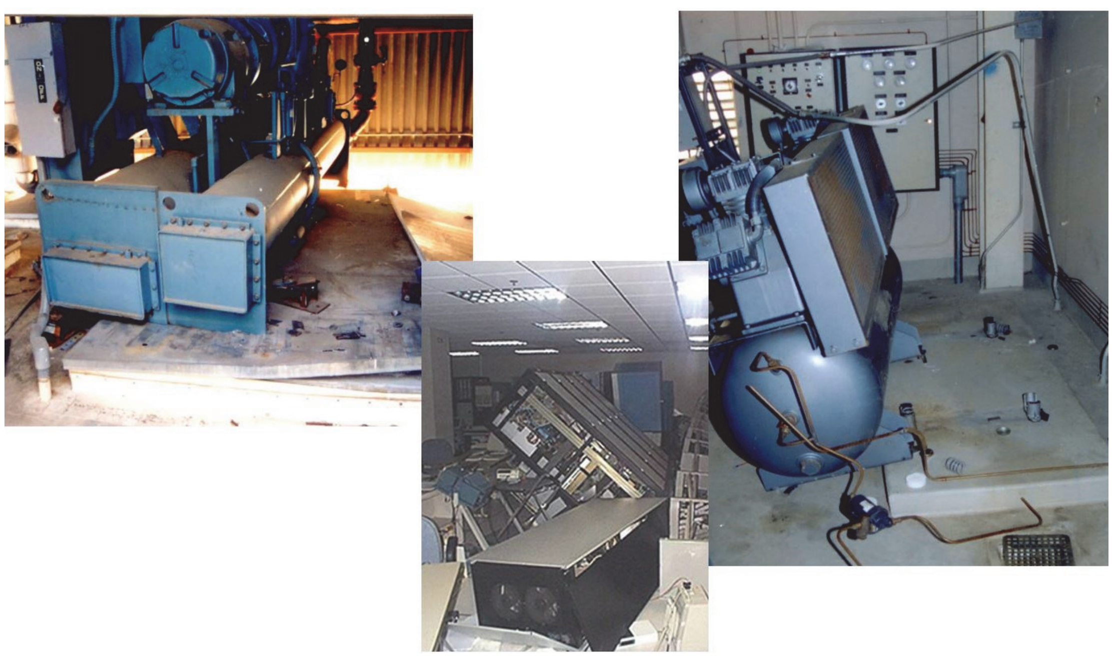


Figure A-7 Equipment anchorage fails, equipment is damaged, and replacement or repair is required.

A.8 Situation No. 8 – Acoustic Tile Ceilings

Figure A-8 Acoustic tile ceiling grid is damaged and requires replacement.

A.9 Situation No. 9 – Exit Doors

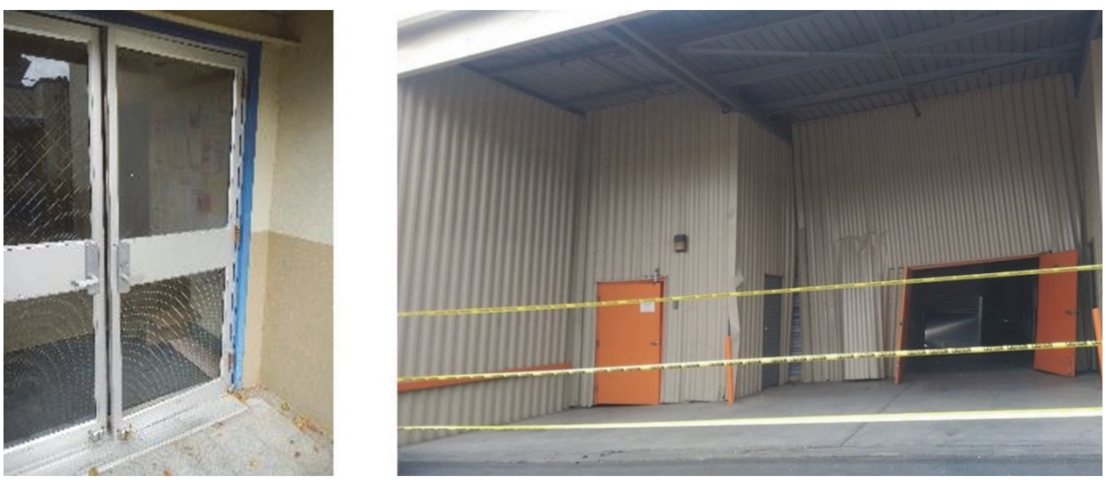
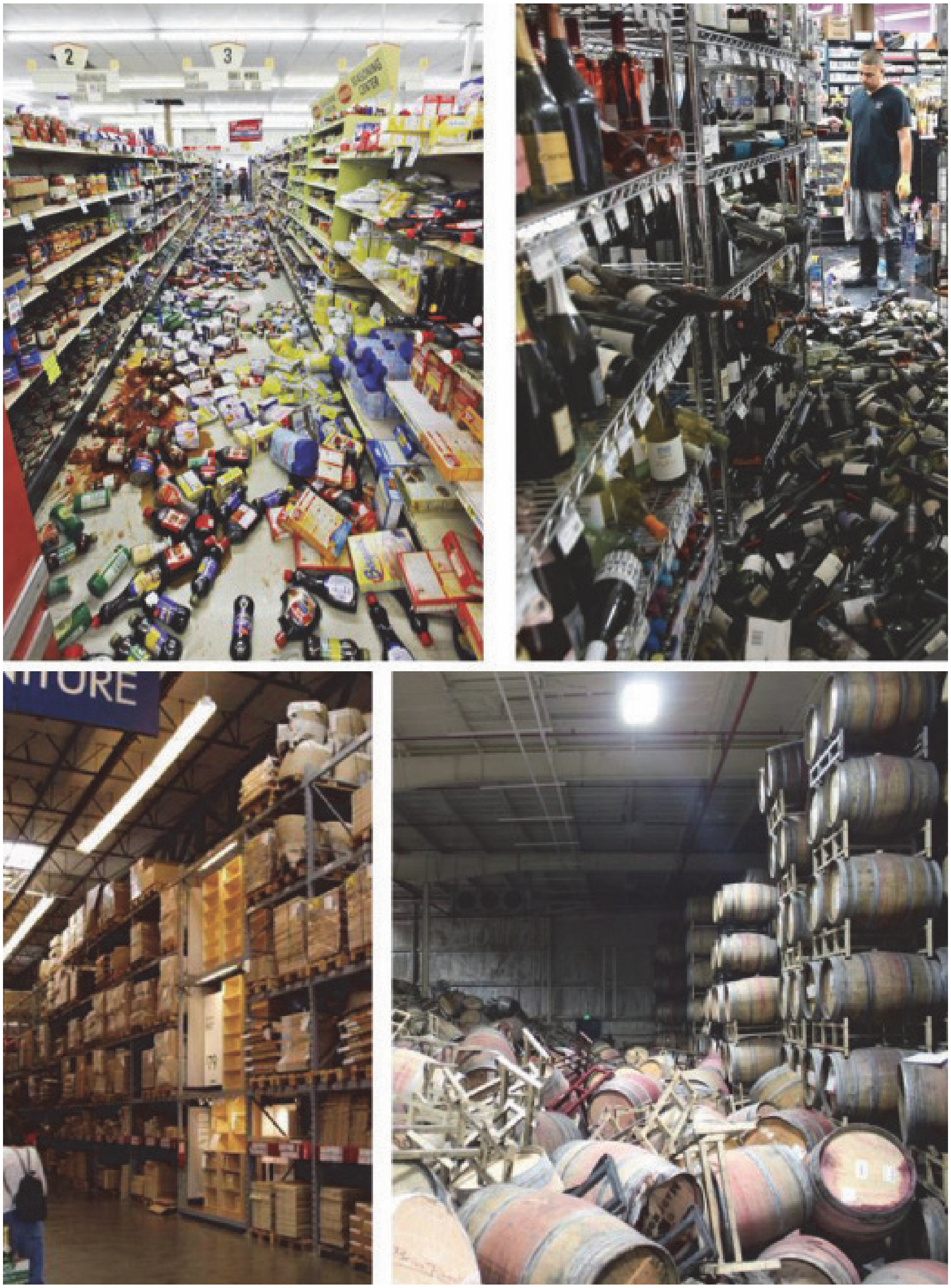


Figure A-9 Exit door is jammed and cannot be opened manually.

A.10 Situation No. 10 – Items on Shelves



Situation 10a: Each item weighs 20 pounds or less and falls from a height of 4 feet or less.
Situation 10b: Each item weighs more than 20 pounds and falls from a height of 4 feet or less.
Situation 10c: Each item weighs 20 pounds or less and falls from a height of 8 feet or more.
Situation 10d: Each item weighs more than 20 pounds and falls from a height of 8 feet or more.

Figure A-10 Items fall from shelving or storage racks in a public facility.

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