# PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

# Guidelines, Specifications, and Seismic Performance Characterization of Nonstructural Building Components and Equipment

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# ABSTRACT

The main objectives of the research project reported herein are to identify gaps in knowledge regarding the seismic behavior of nonstructural building components and to help develop a research strategy within the Pacific Earthquake Engineering Research (PEER) on nonstructural building components.

For this purpose, existing guidelines and regulations for the design and testing (qualification) of nonstructural components were compared, and published analytical and experimental research on nonstructural components was reviewed.

Chapter 1 presents an introduction to this project along with a general discussion on the need to address the seismic behavior of nonstructural building components. In Chapter 2, the performance of nonstructural building components during the recent February 28, 2001, Nisqually, Washington, earthquake is reviewed. The performance of nonstructural components during other earthquakes in the United States is reviewed in Chapter 3. In Chapter 4, an inventory and comparison of existing regulations and guidelines for the seismic design and specification of nonstructural building components are presented. Past analytical and experimental investigations on the seismic response of nonstructural building components are briefly reviewed in Chapter 5. The computerized database that has been developed in this project to centralize this large amount of information, as well as to facilitate any future literature searches on the seismic behavior of nonstructural building components, is briefly described in Chapter 6. Chapter 7 provides a summary of gaps in knowledge and recommendations for the development of a rational research plan on nonstructural building components.

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# 1 Introduction

In many strong earthquakes that have struck the United States in the twentieth century, including the recent February 28, 2001, Nisqually, Washington, earthquake, the damage to nonstructural building components has exceeded the cost of structural damage in most affected buildings (Nonstructural 1984). Architectural components, machinery, and electrical and mechanical equipment mounted within buildings must be designed to withstand the forces and displacements that arise from the seismic response of the structure. Elevators and their counterweights, for example, are vulnerable to large structural displacements as well as to lateral forces. The mounting and support of motors, fans, and other machinery and equipment need sufficient strength to resist the seismic forces transmitted through these components. Also, the failures of interior partitions, finishes, and hung ceilings pose hazards to occupants. With the development of performance-based earthquake engineering, harmonization of the performance levels between structural and nonstructural components is necessary. Even if the structural components of a building achieve an immediate-occupancy performance level during a seismic event, equipment failure inside the building can lower the performance level of the entire building system. This reduction in performance caused by the vulnerability of nonstructural components has been observed in several buildings during the recent 2001 Nisqually earthquake in the Seattle-Tacoma area (Filiatrault et al. 2001).

In comparison to structural components and systems, there is little information available giving specific guidance on the seismic design of nonstructural components for multiple-performance levels. Little basic research has been done in this area and often design engineers are forced to start almost from square one: observe what goes wrong and try to prevent repetitions. This is a consequence of the empirical nature of current seismic regulations and guidelines for nonstructural components. The code information currently available for the most part is based on judgment and intuition rather than on experimental and analytical results. A first comprehensive summary of many important aspects on the seismic behavior of nonstructural elements as well as the evolution of research and code efforts in the twenty years prior to 1995 can be found in Soong (1995).

The research project described in this report is one of two pilot projects funded by the Pacific Earthquake Engineering Research (PEER) Center that have the common objective of determining the state-of-knowledge related to nonstructural building components in order to develop a rational research plan for a coordinated study of nonstructural components within the PEER Center. This project has been coordinated with the companion pilot project conducted by Professor Eduardo Miranda of Stanford University on the limit states and expected performance levels of nonstructural building components.

This project focuses on nonstructural building components, and includes five basic types nonstructural building components:

1. Building contents
2. Building service equipment (equipment required for the normal operation of the building, e.g., electrical system, piping, etc.)
3. Building utilization equipment (equipment introduced into the building for the particular utilization of the building)
4. Interior architectural components
5. Exterior architectural construction

The literature review presented is selective and focuses on references presenting the most up-to-date information on the seismic behavior of nonstructural components.

The project consisted of the following five main parts, namely:

1. The observation of the performance of nonstructural building components during the recent 2001 Nisqually earthquake in the Seattle-Tacoma region;
2. The observation of the performance of nonstructural building components during other past earthquakes;
3. The inventory and comparison of existing regulations (guidelines) for the seismic design and specification of nonstructural building components;
4. The inventory and summary of past analytical and experimental research on nonstructural building components; and
5. The summary of gaps in knowledge and recommendations for the development of a rational research plan on nonstructural building components within PEER.

# 2 Performance of Nonstructural Building Components during the February 28, 2001, Nisqually Earthquake

The Nisqually earthquake struck the Puget Sound in the western region of Washington State on February 28, 2001. The main shock of this $M_w$ = 6.8 seismic event occurred at 10:54 a.m. (PST) and originated at a depth of 52 km. Because of this focal depth, the earthquake caused only light-to-moderate ground shaking in the Puget Sound area. The strong motion duration of the earthquake, however, was relatively long.

A large portion of the estimated $2 billion dollar loss resulting from the Nisqually earthquake was associated with damage to nonstructural components, which makes this seismic event particularly interesting for this project. Even though building structures generally performed well during the earthquake, the inferior performance of nonstructural components reduced the overall performance of many building systems (Filiatrault et al. 2001). In this chapter, the performance of nonstructural building components during the Nisqually earthquake is reviewed separately, since PEER mandated the authors to conduct a field reconnaissance of the performance of nonstructural components immediately following the earthquake. The performance of nonstructural components during other past earthquakes is the subject of the next chapter.

## 2.1 PERFORMANCE OF CEILING SYSTEMS

One of the most common types of nonstructural component failure observed following the Nisqually earthquake was related to suspended ceiling systems. Examples of partial suspended ceiling failure at Sea-Tac Airport are shown in Figs. 2.1 and 2.2.

**Figure 2.1 Partial Failure of Suspended Ceiling at Sea-Tac Airport (Filiatrault et al. 2001).**

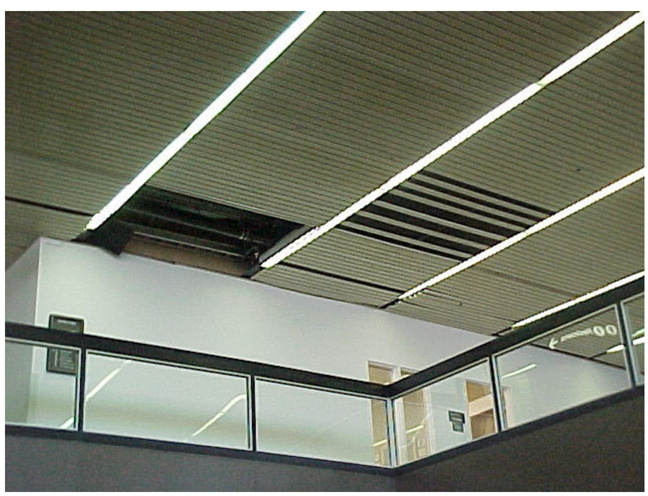


**Figure 2.2 Partial Failure of Metal Suspended Ceiling at Sea-Tac Airport (Filiatrault et al. 2001).**

One of the buildings that experienced the most damage related to suspended ceiling light fixtures was the Starbucks Headquarters building in downtown Seattle. Although the eccentrically braced steel frames used to seismically upgrade the building performed as intended, the suspended lighting fixtures were unable to accommodate the induced lateral acceleration and caused significant damage, as shown in Fig. 2.3. Fortunately, only minor injuries resulted from the failure of suspended lighting fixtures throughout the building. The building also suffered damage caused by the shifting and tumbling of unanchored furniture items and contents, as shown in Fig. 2.4.

**Figure 2.3 Failure of Suspended Lighting Fixtures in Starbucks Headquarters, Seattle (Filiatrault et al. 2001).**

**Figure 2.4 Damage Caused by Unanchored Furniture Items and Building Contents in Starbucks Headquarters, Seattle (Filiatrault et al. 2001).**

## 2.2 PERFORMANCE OF INTERIOR WALL FINISHES

Cracking of interior wall finish materials was observed in many buildings following the Nisqually earthquake. In most cases, diagonal cracking occurred at upper corners of doors and window openings and at the intersection of beams and walls, as shown in Fig. 2.5.

**Figure 2.5 Cracking of Drywall Finish in Beam-to-Wall Connection at Sea-Tac Airport (Filiatrault et al. 2001).**

One interesting observation on the cracking of drywall finish was made at the Kent Regional District Center. Vertical cracking occurred near the upper corner of almost all interior doors of the building. As shown in Fig 2.6, cracks were observed on only one side of each door.

**Figure 2.6 Vertical Cracking of Drywall Finish above Door Opening at Kent Regional District Center (Filiatrault et al. 2001).**

Plaster spalled from the walls and ceilings of the Legislative Building in Olympia, as illustrated in Fig. 2.7. This spalling of plaster above the domed rotunda was one of the concerns that contributed to the closing of the Legislative Building after the Nisqually earthquake.

**Figure 2.7 Spalling of Plaster in Legislative Building, Olympia (Filiatrault et al. 2001).**

Vertical cracking of wall finishes at the corners of perpendicular walls was observed in a number of buildings. Figure 2.8 shows an example of this type of cracking that occurred for the plastered walls of the Supreme Court located in the Temple of Justice Building in Olympia.

**Figure 2.8 Vertical Cracking of Plastered Walls in Supreme Court of Temple of Justice, Olympia (Filiatrault et al. 2001).**

Substantial cracking of interior wall finish materials was observed in the stairwell of the yellow-tagged Olympian Apartments at 519 Washington Street in Olympia, as shown in Fig. 2.9.

**Figure 2.9 Cracking of Plastered Walls in Stairwells of Olympian Apartments, Olympia (Filiatrault et al. 2001).**

Figure 2.10 shows the walls of the original masonry stairwells of the Starbucks Headquarters building in downtown Seattle that suffered severe cracking as a result of the drift level experienced by the building in the east-west direction.

**Figure 2.10 Severe Cracking of Masonry Stairwells in Starbucks Headquarters, Seattle (Filiatrault et al. 2001).**

## 2.3 PERFORMANCE OF EXTERIOR WALL FINISHES

Cracking of exterior wall finish materials was observed in residential wood buildings in Olympia. This cracking was usually diagonal and occurred mainly at the corners of window and door openings, as illustrated in Fig. 2.11 for a three-story apartment building on Columbia Street in Olympia.

**Figure 2.11  Diagonal Cracking of Stucco between Window Openings in Three-Story Apartment Building, Olympia (Filiatrault et al. 2001).**

In some more recent wood-frame houses, wood siding was also damaged as a result of the shaking. Figure 2.12 shows an example of a two-story wood-frame house on Ninth Avenue in Olympia that lost some straight wood siding during the earthquake. Note that the lateral movement of the house caused some boards to be wedged against the rafters.

**Figure 2.12  Damage to Siding Boards on Wood-Frame House, Olympia (Filiatrault et al. 2001).**

## 2.4 PERFORMANCE OF WINDOW SYSTEMS

Shattering of glass windows occurred at several locations. The most dramatic instance of this was the loss of all but one window of the control tower at Sea-Tac airport, as shown in Fig. 2.13. The failures of these windows contributed to the shutdown of the airport for 4 hours following the earthquake because the air-traffic control had to be relocated into a temporary trailer.

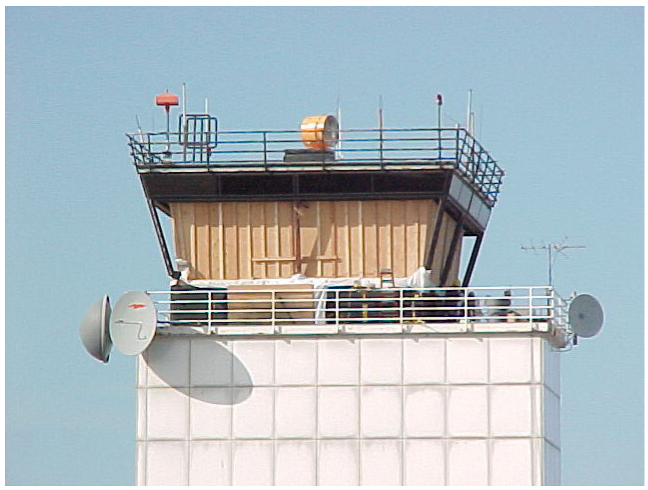


**Figure 2.13 Control Tower Shattered Windows Boarded with Plywood at Sea-Tac Airport (Filiatrault et al. 2001).**

Boarded windows frames were frequently observed in the Pioneer Square area of Seattle. Workers were commonly seen replacing windowpanes in buildings along the streets, as shown in Fig. 2.14.

**Figure 2.14 Replacing Broken Windowpanes in Pioneer Square Area, Seattle (Filiatrault et al. 2001).**

## 2.5 PERFORMANCE OF BUILDING CONTENTS

Another source of damage to nonstructural components during the Nisqually earthquake was the tumbling of building contents. Figure 2.15 shows, for example, the failure of bookshelves that caused some books to fall in the main library of the Temple of Justice Building in Olympia. Figure 2.16 shows a detail of the screwed wood-to-metal connection that failed at the top of one of the leaning bookshelves.

**Figure 2.15 Failed Bookshelves in Main Library of Temple of Justice Building, Olympia (Filiatrault et al. 2001).**

**Figure 2.16 Failed Screwed Wood-to-Metal Connection at Top of Leaning Bookshelf in Main Library of Temple of Justice Building, Olympia (Filiatrault et al. 2001).**

This library contained also sturdier movable compact bookshelves mounted on a floor railing. As shown in Fig. 2.17, these shelves did not suffer any visible damage during the earthquake and seemed to remain functional.

**Figure 2.17 Undamaged Movable Compact Bookshelves and Leaning Conventional Bookshelves in Main Library of Temple of Justice Building, Olympia (Filiatrault et al. 2001).**

Although the leaning bookshelves in the Temple of Justice Building represented an obvious hazard to occupants following the Nisqually earthquake, only a small number of books fell off the shelves, in contrast with what happened in the Law Library of this building. As shown in Fig. 2.18, the massive wood bookshelves of the Law Library did not collapse. A large number of books, however, were thrown to the floor by the horizontal acceleration induced by the ground motion.

Several bookshelves and one unanchored computer also toppled over at the Kent Regional District Center.

**Figure 2.18 Fallen Books from Massive Wooden Shelves in Law Library of Temple of Justice Building, Olympia (Filiatrault et al. 2001).**

The majority of the furniture in the top three floors of the eight-story Ramada Inn at 621 Capitol Way in Olympia was overturned as a result of the building shaking. Figure 2.19 shows typical examples of the damage to furniture in guest rooms on the eighth floor. In the bottom five floors most of the furniture remained in the upright position. None of the room furniture in the building had been fastened to the walls to prevent overturning.

**Figure 2.19 Overturned Furniture in Guest Rooms on Eighth Floor of Ramada Inn, Olympia (Filiatrault et al. 2001).**

## 2.6 PERFORMANCE OF GAS AND WATER SYSTEMS

Only one reported fire erupted at the Cedar Creek Correction Center following the earthquake. One gas shut-off valve was activated at the Kent Regional District Center. It was reported that the residents of 50 mobile homes in Tumwater Mobile Estates were evacuated when a 30-mm gas line ruptured during the earthquake.

Several water lines were severed during the ground shaking. One water line and one chilled water line failed on the fourth floor of the Kent Regional District Center.

A 75-mm-diameter water pipe broke in the mechanical room on the roof of the Ramada Inn at 621 Capitol Way in Olympia, causing 3000 liters of water in a storage tank to flood several floors of the building. During the earthquake the unsecured water tank was reported to have shifted about 150 mm along the floor. This, in turn, caused the supply water line to the tank to rupture as shown in Fig. 2.20.

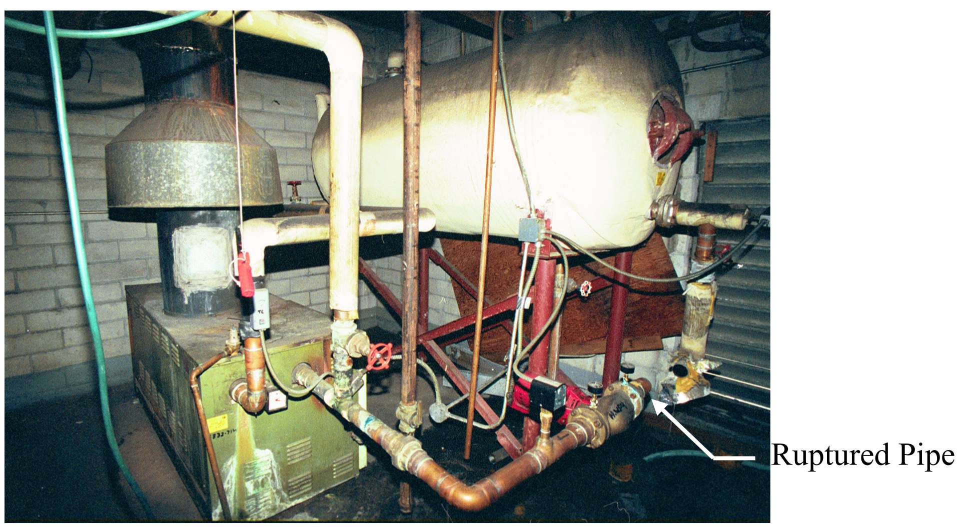



**Figure 2.20 Rupture of Supply Water Line to 3000-Liter Storage Tank in Rooftop Mechanical Room of Ramada Inn, Olympia (Filiatrault et al. 2001).**

# 3 Performance of Nonstructural Building Components during Other Past Earthquakes

Post-earthquake observations and data collections have had the strongest influence in the evolution of seismic design practices for nonstructural components. In almost all earthquakes, it was found that the performance of engineered (or code-conforming) nonstructural components that have been installed properly have been far superior than the performance of nonstructural components installed without any seismic design in mind (Reitherman and Sabol 1995).

In this chapter, a summary of the main observations made on the seismic performance of nonstructural components following major earthquakes that occurred in or are relevant to the United States, other than the recent Nisqually earthquake, are briefly discussed. More complete descriptions can be found elsewhere (Ayres et al. 1973; Ayres and Sun 1973; Ding and Arnold 1990; Reitherman 1994, 1997; Reitherman and Sabol 1995).

## 3.1 PERFORMANCE OF BUILDING CONTENTS

Shaking from a strong earthquake causes unsecured building contents to be shifted or thrown around, raising significantly the hazard level to occupants. Inadequately braced shelving and racks are particularly vulnerable. Observations of residential dwellings after the 1994 Northridge, California, earthquake revealed that kitchens suffered the most contents damage, followed by living and dining rooms (Reitherman and Sabol 1994). Contents damage is not always correlated to the shaking intensity. For example, in the epicentral region of the 1994 Northridge earthquake contents damage to many buildings was surprisingly low (Reitherman and Sabol 1994).

The performance of cantilevered library shelving and storage rack systems during earthquakes in the past 25 years has been poor. Field reports from the 1987 Whittier Narrows, 1989 Loma Prieta, and 1994 Northridge earthquakes indicated that failure of these systems resulted in inventory losses, disruption of operations as well as injuries to occupants. The main reasons for these failures have been identified as (Rihal and Gates, 1998):

- In-plane racking and failure of diagonal rod-bracing connections;
- Out-of-plane failure and collapse due to inadequate anchorage at the base and inadequate overhead lateral bracing between shelving units; and
- Combination of in-plane and out-of-plane damage by torsional response.

Furthermore, it was observed that in cases where the performance of these shelving and storage rack systems was good (i.e., no failures) the material toppling was consistently more significant.

## 3.2 PERFORMANCE OF BUILDING SERVICE EQUIPMENT

### *3.2.1 Performance of Elevator and Escalator Systems*

Elevators are among the most important mechanical systems in building structures and are quite susceptible to earthquake-induced damage. It is estimated that half a million passenger elevators are in service in the United States (Swerrie 1991), many of which are located in active seismic zones. During the 1989 Loma Prieta earthquake, for example, close to 20,000 elevators were located in the region of intense ground shaking (Swerrie 1991). So far elevators have performed very well from the point of view of life-safety performance level, as there are no reported direct fatalities associated with elevator failures in the United States (Suarez and Singh 2000). With respect to the immediate-occupancy performance level, however, elevators remain vulnerable to service disruptions. The main components of an elevator system have been described by Suarez and Singh (2000). For example, in the 1971 San Fernando earthquake 674 cases of derailment of elevator counterweights were reported (Ayres and Sun 1973); in the 1994 Northridge earthquake, 688 (McTiernan 1994).

Damage suffered by elevators during the 1964 Alaska (Ayres et al. 1973), 1971 San Fernando (Ayres and Sun 1973, Sturgeon 1972, Gates and McGavin 1998), 1978 Miyagi (Fukuda 1990), 1986 Carpathian (Nazarova 1990), 1987 Whittier (Schiff 1988), 1989 Loma Prieta (Ding and Arnold 1990, Swerrie 1991), 1994 Northridge (Gates and McGavin 1998; Finley at al. 1996, OSHPD 1995), and 1995 Kobe (Wada and Kitamura 1995) earthquakes has been reviewed by Suarez and Singh (2000). The observed damage can be summarized as follows:

- Damage to guide rail anchorage
- Bent guide rails
- Counterweights dislodging from their guardrails
- Loose counterweights impacting passenger cars
- Control panels tipped or moved
- Traction machines shaken loose from their mountings
- Motor-generator sets shifted across machine room floor
- Ropes damaged by projections or protuberances in the hoist ways
- Suspension ropes jumped from drive
- Seismic switches failed to trigger

Past earthquakes, on the other hand, did not significantly affect escalator systems, until the 1994 Northridge earthquake and the 1995 Kobe earthquake in Japan where damage to many escalators were observed. The fact that both earthquakes occurred early in the morning contributed to the lack of injuries to escalator passengers.

### *3.2.2 Performance of Mechanical, Electrical, and Appliance Equipment*

Large, tall and/or narrow equipment that is not adequately anchored can slide or overturn during an earthquake and cause damage to the equipment itself or to its connections. Mechanical or electrical equipment mounted on vertical vibration isolators can be particularly vulnerable to being shaken off their isolated supports. Suspended equipment swaying during and after an earthquake can cause damage. Unanchored water heaters may slide and overturn and result in broken water and gas lines; the latter representing a significant fire hazard.

During the 1994 Northridge earthquake, mechanical and electrical equipment that was rigidly bolted or anchored to the main structural system performed well, provided that the anchors and supports were designed for code-prescribed loads (Gates and McGavin 1998). On the other hand, equipment mounted on vibration isolation systems such as rubber or springs performed poorly (Reitherman and Sabol 1995, Gates and McGavin 1998). This is mainly due to the unrestrained large displacements that were induced by the ground shaking as well as amplified inertia forces that caused failure of the anchors. As noted by Gates and McGavin (1998), vibration isolation systems are usually designed by mechanical engineers for reducing occupant discomfort under the machine-induced vibrations, and are then simply treated as flexibly mounted elements when computing the seismic forces. These systems had very large dynamic amplification responses that may have exceeded the amplification factors predicted by codes. This highlights also the need for more coordinated efforts among the various specialties involved in the design and installation of nonstructural components.

Damage to mechanical and electrical equipment has been widespread in all past strong earthquakes that have struck the United States in the twentieth century. In the 1994 Northridge earthquake, for example, damage was particularly extensive to spring-isolated mechanical equipment installed in upper stories or roofs of buildings (Reitherman and Sabol 1995).

During the 1994 Northridge earthquake, approximately 2500 water heaters were damaged (Mroz and Soong, 1997). In past California earthquakes, water heaters were a major source of gas leaks, posing an important post-earthquake fire hazard. Of this large number of damaged water heaters, the number of those equipped with some kind of restraints was similar to the number of those without any restraint. This is an indicator that non-engineered restraints were not effective in protecting water heaters.

### *3.2.3 Performance of Ductwork and Piping Systems*

The seismic performance of ductwork and piping systems is of special interest. These systems are expected to remain functional following earthquakes in order to mitigate post-earthquake fire hazard. Also, there is a real potential for significant water damage that can take place if these systems are compromised in a building that did not suffer significant structural damage.

During the 1994 Northridge earthquake, probably the single most disruptive type of nonstructural damage was breakage of water lines inside buildings (Reitherman and Sabol 1995). At least 13 hospitals suffered extensive water damage caused by failures in the pressurized fire sprinkler, and HVAC and domestic water piping systems (Ayres and Philips 1997). Extensive failures occurred in connections in small hot water lines and duct-mounted zone reheat coils. Differential movements between the pipes and the buildings caused also failures of sprinkler systems. The weak link in the fire sprinkler system was identified as being the small branch lines feeding water into the room area within the suspended ceiling (Gates and McGavin 1998). The smaller feeders, typically connected to the main branch by $90^{o}$ bends, experienced large bending stresses because of the interaction of the sprinkler system and the suspended ceiling. In the cases where these bends had been designed to provide a flexible connection no failures were observed.

The most dangerous failure of a piping system occurred when a 12-in. pipe fell from a mall's ceiling, demolishing a kiosk. A surprising observation during the Northridge earthquake is the significant effect of vertical accelerations on sprinkler systems. Many sprinklers were damaged when branch lines moved upward, pushing sprinklers through the ceiling. In general, the lack of bracing or inadequate bracing was cited as a major factor in the most significant failures of fire sprinkler systems during the 1994 Northridge earthquake in California (Fleming 1998) and the 1995 Kobe earthquake in Japan (EQE 1995). Also, failure of unbraced small diameter piping (less than 1-in. diameter) was also common in the Northridge earthquake.

## 3.3 PERFORMANCE OF BUILDING UTILIZATION EQUIPMENT

### *3.3.1 Performance of Emergency Power Systems*

The failure of emergency power systems during an earthquake can be particularly disruptive, since these systems are designed to be activated in the event of an emergency. Emergency power systems include heavy components such as batteries, motor generators, fuel tanks, transformers, switchgear, and control panels that are frequently stored in racks.

The loss of offsite electric power during the 1994 Northridge earthquake put the emergency power supply systems to the test especially for essential operations. The power outage affected over 2 million customers in the Los Angeles area (Reitherman and Sobel 1995). As reported by Merz and Eli (1997), the following observations were made on the performance of emergency power systems after surveying a series of electric power facilities, industrial facilities, power plants, and lifelines after the 1994 Northridge earthquake:

- Emergency generators directly anchored or on engineered isolators with seismic restraints performed well.
- A transfer switch from normal offsite power to emergency power did not function.
- A pumping system transferring fuel from a storage tank to a day tank was non-operational because it was not powered by an emergency power system.
- Failure of a switch from an empty fuel tank to an auxiliary tank caused another emergency generator to be non-functional.
- Electric shorting in electrical enclosures due to water leaks from domestic water and fire sprinklers caused the shutdown of certain power systems.

### *3.3.2 Performance of Hazardous Material Storage Systems*

The failure of hazardous materials supply lines and the improper operation of seismically activated shutoff valves can be life threatening following an earthquake. Toppling of laboratory chemicals must also be prevented during seismic shaking. Tall vertical tanks used for storing fluids are susceptible to overturning under seismic loading when the height to diameter ratio is large. In both the 1989 Loma Prieta and 1994 Northridge earthquakes a number of fluid storage tanks toppled as a result of inadequate anchoring (Gates and McGavin 1998).

## 3.4 PERFORMANCE OF INTERIOR ARCHITECTURAL ELEMENTS

### *3.4.1 Performance of Interior Partitions*

Heavy interior masonry partitions have often failed in past earthquakes due to the excessive flexural out-of-plane stresses or excessive in-plane shear stresses induced by the interstory drifts imposed on the building structure. This type of failure has been observed in numerous earthquakes in the United States, and as far back as the 1925 Santa Barbara earthquake (Dewell and Willis 1925).

### *3.4.2 Performance of Ceiling Systems*

Unbraced suspended ceilings can swing independently of the supporting floor and induce damage, particularly at the perimeters of ceilings. Lay-in ceilings are particularly vulnerable to the relative displacement of the supporting grid members. During the 1994 Northridge earthquake, millions of square feet of ceiling tiles were dislodged along with lighting fixtures and air vent ducts (Gates and McGavin 1998). The effect of the fire sprinkler system that penetrates the ceiling tiles to expose the sprinkler head caused irreparable damage to the tiles while rupturing some of the sprinkler systems, causing subsequent water damage. Recent code changes require spacing between the sprinkler head and the ceiling tile to accommodate the differential movements during seismic loading. Similarly, no spacing is typically provided to accommodate differential movement between the ceiling grid and the perimeter walls. This also contributed to the extensive damage to ceiling systems during the 1994 Northridge earthquake.

### *3.4.3 Performance of Lighting Fixtures*

Fluorescent lighting fixtures that are supported by a suspended ceiling grid can lose their vertical support when the suspended ceiling sways and distorts under ground motion shaking. The splices of electrical wires used to support pendant-mounted lighting fixtures can pull apart causing the fixtures to fall. Lighting fixtures can also swing and impact adjacent objects often causing the fixtures to fall or fail.

Failure of light fixtures was one of the three most frequent kinds of nonstructural damage suffered by school buildings as the result of the 1994 Northridge earthquake (DSA 1994). A new type of lighting fixture damage that was observed during the 1994 Northridge earthquake was the fall from high gymnasium ceilings of high-intensity discharge gas vapor light (Reitherman and Sabol 1995).

### *3.4.4 Performance of Raised Computer Access Floors*

Typical raised computer access floors are constructed of wood, aluminum or steel panels supported on adjustable column pedestals. The columns are often fastened to the sub-floor with mastic. When subjected to lateral loads, raised access floors can be very flexible and can cause an amplification of the ground motion at the base of equipment items supported on the floor. In turn, the base shear induced by these equipment items may cause the raised access floor to collapse.

## 3.5 PERFORMANCE OF EXTERIOR ARCHITECTURAL ELEMENTS

Nonstructural elements used for the exterior construction of a building can suffer significant damage during an earthquake that may result in life-threatening hazards. Observations from past earthquakes have shown that excessive differential motions combined with the lack of lateral strength are responsible for most of the seismic damage suffered by these elements.

### *3.5.1 Performance of Exterior Curtain Walls*

Stiff curtain wall panels attached to the exterior of a building may have insufficient lateral deformation capacity to accommodate the lateral interstory drift imposed on the building by the seismic ground motion input. This problem can be particularly acute when the building is flexible laterally (e.g., a steel moment-resisting frame). Usually failure is observed in the connections between the panels and the building structure.

A review of damage suffered by heavy cladding panels during the 1994 Northridge earthquake indicated that nonstructural cladding panel influenced significantly the performance of several buildings, and that efforts should be undertaken to provide proper engineering details to these elements that currently are ignored in design (Cohen 1995).

A study by Goodno et al. (1989) correlated damage to heavy cladding components observed during the 1985 Mexico earthquake to experimental and analytical results. It was found that in many cases, claddings increased the initial stiffness of the building before suffering extensive damage. This stiffening effect is usually not taken into account during the design process, and may considerably influence the dynamic response of the building. It was also noted that although the damaged cladding systems were replaced following the earthquake, the engineering effort to fully assess the retrofit or repair was minimal. As a result of this, the authors believe that many of the connections of the replaced or repaired cladding systems to the main structural system may be inadequate.

### *3.5.2 Performance of Exterior Veneers*

Stone and masonry veneers with inadequate anchorage have often failed in past earthquakes. A most unfortunate example of this occurred during the 1987 Whittier Narrows earthquake in California when a precast concrete panel fell from a parking structure and killed a student (Taly 1988).

Out-of-plane failure of precast cladding, brick and masonry veneers was widespread in the epicentral region of the 1994 Northridge earthquake (Reitherman and Sabol 1995). Goodno

(1994) provides a survey of the damage suffered by heavy cladding systems during this earthquake.

### *3.5.3 Performance of Glass Doors, Windows, and Glazing*

The principal cause of glass door and window failures during earthquakes is the inadequate edge clearances around the glass to allow the building to deform laterally without bearing on the glass. Glazing damage was extensive during the 1994 Northridge earthquake. In some cases glazing damage was so severe that the supporting metal frames buckled. In some areas, however, only a small proportion of buildings that suffered glazing damage were red-tagged (Reitherman and Sabol 1995). These buildings could have been hazardous in the event of a strong aftershock.

Low-rise buildings that incorporated annealed glass (rather than tempered, wired, or laminated glass required for taller buildings) produced sharp-edged pieces that could have caused serious injuries. Film-coated windows, on the other hand, performed well (Reitherman and Sabol 1995). Window breakage due to flying contents was also observed. Highrise glazing generally performed well during the Northridge earthquake. An industry survey after the earthquake revealed that glazing incorporating silicon sealant performed better than glazing with vinyl gaskets (Harter 1994, Vallabhan 1994). Systems equipped with Mylar film to provide seismic protection from sharp glass debris performed very well in the case of small windowpanes, but proved less effective for larger ones, where the entire pane was dislodged and fell as one big piece (Gates and McGavin 1998).

### *3.5.4 Performance of Chimneys*

The vulnerability of brick chimneys has been demonstrated in all strong earthquakes that struck the United States in the twentieth century. Even tall precast concrete chimneys used in recent residential construction were damaged during the Northridge earthquake by rotating at their base and separating themselves from the main structure (Reitherman and Sabol 1995).

# 4 Inventory and Comparison of Existing Regulations and Guidelines for the Seismic Design and Specification of Nonstructural Building Components

Before 1961 the Uniform Building Code (UBC) did not contain any specific seismic design requirements for nonstructural components. The 1961 edition of the UBC introduced a seismic force analysis procedure applicable to nonstructural building components. The 1964 Alaska and the 1971 San Fernando earthquakes demonstrated for the first time that damage to nonstructural building components could result in casualties and injuries, disruption of building operation, and significant economic losses (Lagorio 1990).

In the last decade, a number of federal agencies, including the United States Postal Service, have developed and implemented guidelines for the seismic evaluation and retrofit of nonstructural building components (Applied Technology Council 1992a, 1992b). A summary of recent developments in codes for nonstructural components can also be found in Soong (1994a).

## 4.1 SEISMIC PERFORMANCE REQUIREMENTS FOR NONSTRUCTURAL BUILDING COMPONENTS

Table 11-1 of the *NEHRP Guidelines for Seismic Rehabilitation of Buildings* and its commentary (ATC 1997a, 1997b) lists requirements for the applicability of life-safety, immediate-occupancy requirements, and methods of analysis for a variety of architectural, mechanical, and electrical building components. These performance requirements are established in relation to three different seismic zones. The NEHRP guidelines address only components that are permanently installed in buildings. Other nonstructural components, such as building contents introduced by owners, are not covered by the guidelines. The NEHRP guidelines define four different performance levels for nonstructural building components:

1. **Reduced-Hazards Performance Level:** Extensive Damage but prevention of immediate falling hazard from heavy items.
2. **Life-Safety Performance Level:** Prevention of falling hazard from all items that can directly result in injury.
3. **Immediate-Occupancy Performance Level:** No falling hazard, minor damage and disruption to nonstructural components, but the building can be occupied.
4. **Operational Performance Level:** Continuing operation of all nonstructural components.

## 4.2 DESIGN FORCE AND DRIFT REQUIREMENTS

Several seismic design force requirements have been developed in the United States for nonstructural components. These provisions have evolved mainly from those included in the Uniform Building Code (UBC) and those developed as part of the National Hazard Reduction Program (NEHRP). The NEHRP provisions have been developed on a strength base, while, until the 1997 edition of the UBC, the UBC provisions have been based on allowable stress design. In this section, the current seismic design force and drift requirements in the United States are reviewed. Comparative studies on the application of these requirements can be found elsewhere (Soong 1994, Singh et al. 1993, Freeman and Kehoe 1997, Phan and Taylor, 1996, Taylor and Phan 1997, Backman and Drake 1998, Drake and Bragagnolo 2000).

### *4.2.1 The SEAOC Blue Book (1996)*

The SEAOC Blue Book (SEAOC 1996), prepared by the Structural Association of California, reproduces section 1630 of the 1994 edition of the Uniform Building Code (ICBO 1994) and describes the lateral forces that must be applied on elements of structures, nonstructural components, and equipment supported by structures. The 1994 UBC provisions are based on allowable stress design. The allowable stress design seismic force on building parts, $F_p$, specified in the SEAOC Blue Book is given by:

$$F_p = Z\, I_p C_p C_f C_g W_p \tag{4.1}$$

where

$Z$ = Seismic zoning factor, 0.4 in Zone 4

$I_p$ = Importance factor, 1.0 except for essential or hazardous parts where it takes a value of 1.5

$C_p$ = Seismic coefficient for parts, function of nonstructural element flexibility, energy dissipation capacity, and location in building, value varies from 0.75 to 2.0

$C_f$ = Flexibility coefficient, 1.0 for rigid equipment (period less than 0.06 s) and 2.0 for flexible equipment

$C_g$ = At-grade coefficient, 1.0 for flexible parts above grade, 0.67 for parts laterally supported or below grade

$W_p$ = Operating weight of the part

Equation (4.1) assumes constant floor acceleration along the height of the building.

### *4.2.2 The Uniform Building Code (1997)*

Section 1632 of the 1997 edition of the Uniform Building Code (ICBO 1997) provides lateral force requirements for elements of structures, nonstructural components, and equipment weighing more than 400 lbs and supported by structures. The 1997 UBC provisions are based on strength design. The strength design seismic force on building parts, $F_p$, specified in the SEAOC Blue Book is given by:

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \tag{4.2}$$

with

$$0.7 C_a I_p W_p \le F_p \le 4 C_a I_p W_p \tag{4.3}$$

where

$a_p$ = Component amplification factor, function of nonstructural element flexibility, varies from 1.0 to 2.5

$C_a$ = Seismic coefficient, function of the seismic zone factor and soil type

$h_x$ = Element or component attachment elevation with respect to grade

$h_r$ = Structure roof elevation with respect to grade

$R_p$ = Component response modification factor, function of nonstructural element energy dissipation capacity, varies from 1.5 to 3.6

$I_p$ = Importance factor, varies from 1.0 to 1.5

$W_p$ = Operating weight of part

Equation (4.2) assumes a linear variation of floor acceleration along the building height. Alternatively, the UBC allows the use of the upper bound values in equation (4.3), resulting in a constant floor acceleration along the building height.

### *4.2.3 The NEHRP Guidelines for Seismic Rehabilitation of Buildings (1997) and the International Building Code (2000)*

The *NEHRP Guidelines for Seismic Rehabilitation of Buildings* and its commentary (ATC 1997a, 1997b) describe an analytical and a prescriptive seismic analysis procedure for nonstructural building components. For this purpose, the NEHRP guidelines define two types of nonstructural building components: *acceleration-sensitive* components and *deformation-sensitive* components. The procedure to determine in which category a particular nonstructural component belongs depends on the level of inertia forces that can be generated by the component during lateral shaking, and can be somewhat subjective.

Acceleration-sensitive components must resist a strength-based seismic design force, $F_P$, distributed proportionally to its mass distribution given by:

$$F_p = \frac{0.4 a_p S_{XS} I_p W_p \left(1 + \frac{2x}{h}\right)}{R_p} \tag{4.4}$$

with

$$0.3 S_{XS} I_p W_p \le F_p \le 1.6 S_{XS} I_p W_p \tag{4.5}$$

where

$a_p$ = Amplification factor, related to the rigidity of the component (value of 1.0 or 2.5)

$S_{XS}$ = Spectral response acceleration at short periods for the design hazard level

$h$ = Average roof elevation of structure relative to grade level

$I_p$ = Component performance factor, 1.0 for life safety or 1.5 for immediate occupancy

$R_p$ = Response modification factor related to the ductility of the anchorage, (from 1.25 to 6.0)

$W_p$ = Operating weight of the component

$x$ = Elevation of component in the structure relative to grade level

Equation (4.4) assumes a linear variation of floor acceleration along the building height. Alternatively, the UBC allows the use of the upper bound values in equation (4.4), resulting in a constant floor acceleration along the building height.

Also, NEHRP guidelines allow the use of an alternative prescriptive procedure when published standards and references describing the design concepts and construction features are available for a particular building components. In this case no engineering calculations are required.

When drift is also a consideration for acceleration-sensitive components and for deformation-sensitive components, the NEHRP guidelines also require calculating the relative seismic displacement for which the component must be designed. For two connection points located on the same building or structural system the design drift ratio, $D_r$, is given by:

$$D_r = \frac{\delta_{xA} - \delta_{yA}}{X - Y} \tag{4.6}$$

For two connection points at the same level on separate buildings or structural systems, the relative seismic displacement, $D_p$, that the component must be designed to accommodate is given by:

$$D_p = |\delta_{xA}| + |\delta_{xB}| \tag{4.7}$$

where

$X$ = Height of the upper support attachment at level x relative to the grade level

$Y$ = Height of the upper support attachment at level y relative to the grade level

$\delta_{xA}$ = Deflection of Building A at level x

$\delta_{yA}$ = Deflection of Building A at level y

$\delta_{xB}$ = Deflection of Building B at level x

The International Building Code (IBC 2000) adopted the design seismic forces and displacement equations of the 1997 NEHRP guidelines.

The 1997 UBC and the 2000 IBC specify load combinations for nonstructural elements that include the effect of both horizontal and vertical ground motions. The basic load combinations are given by:

$$U \geq 1.2D + 0.5L + E_h + E_v \tag{4.8}$$

and

$$U \geq 0.9D - E_h - E_v \tag{4.9}$$

where

U = Strength Capacity

D = Dead load

L = Live load

$E_h$ = Horizontal seismic force

$E_v$ = Vertical seismic force

The horizontal seismic force $E_h$ is taken as the seismic force $F_p$ defined by (4.2) and (4.3) for the 1997 UBC and by (4.4) and (4.5) for the 2000 IBC. The vertical force $E_v$ is given by:

$$E_v = 0.5 C_a W_p \tag{4.10}$$

for the 1997 UBC, and

$$E_v = 0.2 S_{DS} W_p \tag{4.11}$$

for the 2000 IBC, and where

$C_a$ = Seismic site coefficient

$S_{DS}$ = Design spectral acceleration

The 1997 UBC and the 2000 IBC seismic design forces for nonstructural components are very similar for all levels of seismicity and soil types (Bachman 1998).

#### *4.2.4 The TRI-Services Manual "Seismic Design Analysis for Buildings" (WJE 1996)*

Design guidelines for nonstructural components were first introduced in the 1982 edition of the Tri-Services Manual. The latest edition, prepared in 1996 for the U.S. Army Engineering Division by Wiss, Janney, Elstner Associates, Inc. (WJE 1996) conforms generally to the Structural Engineers Association of California (SEAOC) recommendations of 1990 (Freeman, 1998), but differs in certain aspects.

The introduction of amplification factors for non-rigid or flexibly supported equipment to account for the dynamic response of nonstructural components to building motion, as well as the elaboration of dynamic analyses procedures that account for both the elastic and inelastic seismic response of the building are the two major additions to the SEAOC (1990) recommendations. In addition, the document considers two levels of seismic input and incorporates performance requirements, methods for estimating floor response spectra as well as design examples.

### 4.3 DESIGN GUIDELINES FOR BUILDING CONTENTS

Hillman, Biddison & Loevenguth (1977) have produced guidelines for the seismic restraints of kitchen equipment complying with the 1976 edition of the Uniform Building Code. Typical detail drawings were provided for 19 different basic kitchen equipment categories.

### 4.4 DESIGN GUIDELINES FOR BUILDING SERVICE EQUIPMENT

Damage to building service equipment can compromise the operation of a building immediately after an earthquake. This interruption in operation can be detrimental to life safety for essential buildings such as hospitals.

The FEMA-172 handbook (BSSC 1992) presents simple techniques for mitigating the potential seismic damage that can occur to common building equipment including: elevator and escalator systems, mechanical and electrical equipment, ductwork and piping, emergency power systems, hazardous material storage systems and computer equipment.

In the mid-1980s, the Electric Power Institute initiated a project to evaluate the seismic safety of nuclear power plants by collecting and evaluating existing qualification test data (Smith and Merz 1985). In the first phase of the project, 7 different equipment classes were considered. In the second phase, the methodology was extended to approximately 20 other classes of equipment. The results of the study allowed the generation of generic ruggedness spectra for each specified equipment class.

Recently Johnson et al. (1999) developed a detailed methodology for the assessment and improvement of the functional reliability of equipment systems in critical facilities, such as hospitals, following earthquakes. The methodology is designed to be used by regular staff members, and is based on a simple and rapid assessment of equipment items. The implementation of the methodology requires four major steps: (1) the systems required for life-safety purposes need to be identified; (2) a rapid visual screening needs to be performed on each individual equipment item to determine a relative score; (3) logic diagrams must be used to develop overall scores for all systems based on the scores of individual equipment items; system vulnerabilities can be identified based on these scores and remedial actions can be determined; and (4) the results of the evaluation must be implemented to elaborate a risk management strategy.

### *4.4.1 Specific Design Guidelines for Elevator and Escalator Systems*

The ASME A17.1 Safety Code for Elevators and Escalators (American Society of Mechanical Engineers 1996) is used in the United States to mitigate the potential earthquake-induced damage to elevators and to enhance their seismic performance.

For seismic zones 2 and higher, the A17.1 Code requires the installation of upper and lower position restrainers to the cars and counterweight frames of elevator systems. The purpose of these restrainers is to prevent the counterweight and car from disengaging from the rail if the roller guides fail in a seismic event (Suarez and Singh 2000). The specifications of the A17.1 Code require that the guide rails of elevators be constructed of T-sections conforming to prescribed weights and dimensions. Part XXIV of the A17.1 Code includes graphs for the selection of the minimum bracket spacing for each of the prescribed T-sections. These curves are based on a horizontal seismic force of 0.5 g that should not cause any damage to guide rails. Part XXIV of the A17.1 Code also provides design formulas to determine the maximum allowable weight per pair of guide rails. The formulas depend on the direction of the applied seismic forces. The code does not provide any commentary discussing the theoretical background for these formulas. For seismic zones 2 and 3, the seismic section of the A17.1 Code also requires that the guide rails possess a minimum moment of inertia. Part XXIV of the A17.1 Code provides a set of formulas to calculate the design forces for the guide rail brackets.

The A17.1 Code requires two "fail-safe" earthquake protective devices for all traction elevators operating at a speed of 150 ft/min and above: a *seismic switch* and a *displacement switch*. A seismic switch is a mechanical or electromechanical device activated by a given threshold ground motion to provide a signal that a potentially damaging earthquake is imminent. The A17.1 Code requires that upon activation of a seismic switch, cars in motion must proceed to the nearest floor, open their doors, and shut down. A displacement switch is a device actuated by the displacement of the counterweight at any point on the hoist way to provide a signal that the counterweight has been displaced from its normal operating plane of travel or has left its guide rails. The A17.1 Code requires that upon activation of a displacement switch, the cars in motion must stop and then proceed to the nearest landing at reduced speed in the direction away from the counterweight.

At the time of writing, the current edition of the A17.1 code did not include any seismic requirements for escalators or moving walkway systems. Recently, new seismic requirements for escalators have been added in the 1998 edition of the California Elevator Safety Construction Code. Suarez and Singh (2000) summarized these requirements as follows:

- The connections and beam seats between the escalator/walkway system and the building structure must be designed for a horizontal seismic acceleration of 0.5 g, and must also accommodate twice the interstory drift of the building structure in both directions.
- The handrail supports must be able to sustain a design load of 150 lbs/ft applied at the top of the handrail.
- At least one seismic switch must be provided in any building containing escalators and/or moving walkways. All escalators and/or moving walkways must stop upon activation of the seismic switch.
- Seismic restraints must be provided in the longitudinal direction at the ends of an escalator.
- Seismic restraints must be provided at all supports in the transverse direction.

### *4.4.2 Specific Design Guidelines for Mechanical, Electrical, and Appliance Equipment*

Applicants for an operating license for a nuclear power plant in the United States are required to submit to the Nuclear Regulatory Commission (NRC) a final safety analysis report. This report must include a seismic qualification review of the plant and its components, including mechanical and electrical equipment. To provide guidance to nuclear power plant applicants, the NRC has published guidelines to conduct seismic and dynamic qualification of electrical and mechanical equipment for nuclear power plants (Subudhi et al. 1986). These guidelines present generic information about the dynamic environment and equipment mounting simulations, procedures to conduct dynamic qualifications by analysis and/or testing, and the NRC evaluation procedure.

### *4.4.3 Specific Design Guidelines for Ductwork and Piping Systems*

The 1997 UBC includes recommendations for piping systems. The design lateral seismic force $F_p$ is given by:

$$F_p = 0.56 C_a I W \tag{4.12}$$

where

$C_a$ = Coefficient depending on supporting structure foundation type and seismic zone factor

I = Importance factor equal to 1 for non-essential piping and 1.25 for hazardous or essential piping

W = Deadweight of pipe

The 2000 IBC has introduced more elaborate and complex requirements for piping systems. The design seismic force $F_p$ is bounded by:

$$0.3 S_{DS} I_p W_p \leq F_p \leq 1.6 S_{DS} I_p W_p \tag{4.13}$$

where

$S_{DS}$ = Spectral response at short periods depending on foundation conditions and seismic zone intensity

$I_p$ = Importance factor, equal to 1 for non-essential piping and 1.5 for hazardous or essential piping

$W_p$ = Operating weight of the pipe

The ASCE 7-95 published by the American Society of Civil Engineers also contains requirements for seismic design of piping. These requirements are very similar to the 2000 IBC.

Two methods are generally used for the design of pipes in practice (Stevenson 1998). The "design by rule method" consists of determining the spacing between piping supports to implicitly assure that the stresses and deformations in the supports and piping are within the allowable limits. One such method has been published by the Electric Power Research Institute (EPRI 1990). In the "design by analysis method" the loads on the supports and stress resultants on the piping are computed by applying the seismic forces in combination with other loads and by comparing to allowable stress values or force resistance to carry out the design. Typically, the "design by rule method" is used for small-diameter piping and for areas of low seismicity.

Factors that typically influence the seismic design of pipes are (Stevenson 1998):

- Location of the facility, with respect to seismic zones
- Pipe size
- Pipe classification, i.e., normal, hazardous, or essential

A piping system is defined hazardous or essential if:

- It contains hazardous materials
- The operating temperature is above $650^o$ C, or

- It must perform an essential safety-related function during or immediately following an earthquake

The Sheet Metal and Air-Conditioning Contractors National Association (SMACNA 1992) has published guidelines for the design of seismic restraints of new mechanical systems and plumping piping systems in areas of high seismicity. These guidelines can also be used for the rehabilitation of existing systems. The SMACNA guidelines for the seismic bracing of *ductwork* can be summarized as follows:

- Seismic braces are required for rectangular ducts that have an area of 6 $ft^2$ and greater and for circular ducts that have a diameter of 28 in. or larger.
- Transverse braces should be installed at a maximum of 30 ft on center, at each duct turn, and at each end of a duct run.
- Longitudinal braces should be installed at a maximum of 60 ft on center.
- No bracing is required if the top of a duct is suspended 12 in. or less from the supporting structural member and the suspension straps are attached to the top of the duct.

The SMACNA guidelines for the seismic bracing of *piping* can be summarized as follows:

- Seismic braces are required for all pipes that have a diameter of 2.5 in. and larger and also for smaller piping used for fuel gas, oil, medical gas, compressed air and/or located in boiler rooms, mechanical equipment rooms, and refrigeration machinery rooms.
- Transverse braces should be installed at a maximum of 40 ft on center.
- Longitudinal braces should be installed at a maximum of 80 ft on center.

The Sheet Metal and Air Conditioning Contractors National Association (SMACNA 1980, 1985) also provides prescriptive design methods for strengthening the supports and bracing of HVAC and special ductwork systems.

The SP-58 (MSS 1993) document of the Manufacturers Standardization Society of the Valve and Fitting Industry includes prescriptive methods for the strengthening of support and bracing of piping systems.

Vagliente et al. (1986) pointed out the need for guidelines related to the seismic performance of plastic piping, since plastic materials are now being used to replace ductile steel and copper as a basic piping material. The lateral support of plastic piping must take into account the reduced strength of plastic compared to steel and copper.

Prescriptive seismic design approaches to support bracing of fire suppression piping systems are given in the National Fire Protection Association standard NFPA-13 (NFPA 1996). Fleming (1998) described the historical development of the NFPA-13 seismic requirements for fire sprinkler systems and proposed modifications for the future editions of the NFPA standard.

## 4.5 DESIGN GUIDELINES FOR BUILDING SERVICE EQUIPMENT

### *4.5.1 Specific Design Guidelines for Electrical and Communication Systems*

The Sheet Metal and Air Conditioning Contractors National Association (SMACNA 1980, 1985) provides prescriptive guidelines for strengthening the supports and bracing of electrical and communication conduits.

Reitherman and Minor (1989) published technical guidelines for the seismic protection of typical nonstructural component items encountered in communication facilities. These guidelines present typical details for the attachment of equipment to walls, the anchoring of equipment to concrete slabs, the anchoring of cabinet and box-like equipment, the anchoring of equipment racks, the bolting of adjacent cabinets for increased stability, the restraining of countertop equipment, and the installation of safety wires for overhead light fixtures. The guidelines also present alternatives to the anchorage of equipment by taking advantage of controlled sliding and rolling. A less technical companion pamphlet on the same subject was also published (Reitherman 1987).

Parise et al. (1995) discussed the design and installation criteria to insure the functional reliability of electrical power systems in buildings. In particular, the authors emphasize the use of the "brush-distribution" approach to reduce drastically the seismic vulnerability of electrical power systems in buildings. In this approach, the heaviest electrical equipment items (transformers, generators, motors, panel boards, etc.) are to be located, as much as possible, in ground or underground floors. Also, the electrical distribution in the upper floors of buildings should be subdivided into vertical sectors (or towers) along the height of the building.

Chapter 4 of the "*Network Equipment-Building System (NEBS) Requirements: Physical Protection*" published by Bellcore (1995) provides generic criteria for the performance of telecommunications equipment during earthquakes. In Chapter 5 of the same document, a dynamic procedure for testing equipment located in seismic zone 4 is presented. For lower seismic areas, the accelerations are scaled down. On a shake table the equipment is subjected to a synthetic waveform developed from a series of historical records in the three orthogonal directions to assess the seismic performance level. Static pull tests can also be used to provide approximations of framework strength and stiffness characteristics. It is required that equipment withstands the prescribed waveform without sustaining any permanent structural or permanent damage. It is also required that the maximum displacement at the top of the framework containing the equipment be limited to three inches. Requirements on the natural frequency of certain equipment are also presented.

### *4.5.2 Specific Design Guidelines for Computer Equipment*

The seismic design or rehabilitation of computer equipment is different from that required for other mechanical and electrical equipment for the following reasons:

- Computer equipment is often installed on raised access floors that complicate anchorage interventions and can amplify seismic loads at the base of the equipment.

- Computer equipment is evolving rapidly causing frequent replacement of equipment.
- The electronic components of computer equipment can be damaged by shaking.

Olson (1987) provided detailed recommendations for the seismic design and rehabilitation of computer equipment. These techniques, however, apply mainly to the mainframe type of computer equipment that is now being largely replaced by desktop computers.

## 4.6 DESIGN GUIDELINES FOR INTERIOR ARCHITECTURAL ELEMENTS

The FEMA-172 handbook (BSSC 1992) presents techniques for the seismic design and retrofit of a variety of architectural elements used for the interior construction of buildings, including interior partitions, ceiling systems, lighting fixtures and computer access floors. Most techniques are illustrated with sketches, and the relative merits of the techniques are discussed.

### *4.6.1 Design Guidelines for Interior Partitions*

Chapter 25 of the 1997 edition of the Uniform Building Code (ICBO 1997) presents guidelines for taking into account the in-plane shear resistance of gypsum board and plaster for the seismic design of wood-frame structures. The allowable shear resistance for vertical diaphragms of lath and plaster or gypsum board frame wall assemblies to be used for seismic design is given in Table 25-I of the Code. These values can be used only if the height-to-length ratio of the wall section does not exceed 2 to 1. Blocking must be used if the height-to-length ratio of the wall section does not exceed 1.5 to 1.

### *4.6.2 Design Guidelines for Ceiling Systems*

Following the damage sustained by nonstructural components during the 1994 Northridge earthquake, the City of Los Angeles created working groups to look into possible amendments to building ordinances. These studies were carried out with the active participation of the City of Los Angeles, the Division of State Architects, engineers, architects, property owners, academic researchers, and industry representatives. The major proposed changes are (McGavin et al. 1998):

- Limit of six vertical feet to suspended ceiling without requiring the system to be engineered.
- Braces to the structure at changes in ceiling plane elevation or at discontinuities in the ceiling grid.
- Cable trays and other electrical conduit or piping to be independently supported.
- Oversizing of 1/2 in. required in fire sprinkler head penetrations for independent movement between the ceiling and the sprinkler
- All lay-in panels within a 4-ft radius of exit lights/signs shall be held in place by panel hold-down clips.

## 4.7 DESIGN GUIDELINES FOR EXTERIOR ARCHITECTURAL ELEMENTS

The FEMA-172 handbook (BSSC 1992) presents techniques for the seismic design and retrofit of a variety of nonstructural elements used for the exterior construction of buildings including: curtain walls, appendages, and exterior veneers. Most techniques are illustrated with sketches, and the relative merits of the techniques are discussed.

### *4.7.1 Design Guidelines for Exterior Veneers*

Typical details for approved anchorage of masonry veneers are published by the Brick Institute of America.

### *4.7.2 Design Guidelines for Glass Doors, Windows, and Glazing*

Bouwkamp and Meehan (1960) conducted early racking tests on window frames in order to provide guidance for public school design.

Reitherman (1985) recommended a technique to reduce the life-safety hazard caused by falling glass during earthquakes. The technique involves applying adhesive solar film to the glass windows and doors. The film is intended to hold together the glass fragments while also reducing the heat and glare.

# 5 Inventory and Summary of Past Analytical and Experimental Research on Nonstructural Building Components

Compared to structural components and systems, analytical and experimental investigations on the seismic performance of nonstructural components have been scarce and have not significantly influenced the development of codes and regulations. The only exception has been in the case of nuclear power plants, for which there has been a major investment in testing and analysis.

In this chapter, analytical and experimental investigations on the seismic response of nonstructural building components are briefly reviewed. It can be easily observed that the analytical work has largely exceeded experimental studies and field observations. Only after the 1989 Loma Prieta (Rihal 1992) and 1994 Northridge (URS Consultants/John A. Blume & Associates 1996) earthquakes were systematic studies undertaken to try to combine observations of nonstructural damages with detailed structural analyses. Hopefully this trend will continue in the future.

## 5.1 PAST STUDIES ON DESIGN FORCE REQUIREMENTS

The development of rational methods for the seismic analyses of nonstructural components attached to building structures started over 30 years ago with a particular focus on critical equipments contained in nuclear power plants. In this section, the various analysis methods proposed that have led to the determination of seismic design forces on nonstructural components are briefly reviewed. More detailed information can be found elsewhere (Chen and Soong 1988, Singh 1990, Soong 1994, Villaverde 1997b).

One of the first simplified methods developed to determine the seismic design forces on nonstructural components attached to a building structure is the so-called floor response spectrum technique. In this approach, the acceleration time-history at the base of the nonstructural component in the structure is obtained by a time-integration analysis. The acceleration response spectrum of this acceleration time-history is then computed to obtain a floor response spectrum from which spectral acceleration demand on the nonstructural building component can be obtained. The direct computation of floor response spectra can be tedious, since it requires lengthy step-by-step analyses. For this reason, several approximate procedures have been proposed to generate directly floor response spectra (Biggs and Roesset 1970, Amin et al. 1971, Kapur and Shao 1973, Peters et al. 1977, Vanmarcke 1977, Atalik 1978, Singh 1980).

The floor response approach, however, has three major limitations: (1) the dynamic interaction between the nonstructural components and the building structure is neglected, (2) nonstructural components that have multiple attachment points along the building height can not be properly considered, and (3) floor response spectra are valid only for linear systems. In view of these limitations, more sophisticated approaches based on empirical corrections of linear floor response spectra and/or random vibration analyses have been elaborated (Newmark 1972, Shaw 1975, Thailer 1976, Kawakatsu et al. 1979, Sackman and Kelly 1979, Newmark and Villaverde 1980, Viti et al. 1981, Der Kiureghian et al. 1983, Lee and Penzien 1983, Hernried and Sackman 1984, Gupta 1984, Igusa and Der Kiureghian 1985a 1985b, Lin and Mahin 1985, Singh and Sharma 1985, Villaverde 1986, Asfura and Der Kiureghian 1986, Gupta and Jaw 1986, Burdisso and Singh 1987, Suarez and Singh 1987a 1987b, Villaverde 1987, Igusa 1990, Muscolino 1990, Villaverde 1991).

Schroeder and Bachman (1994) performed nonlinear time-history dynamic analyses to evaluate the effects of the dynamic characteristics of primary structures on the seismic response of secondary systems. This study was performed to provide background materials for the development of the 1994 NEHRP provisions for nonstructural components (ATC 1994). The results indicated that nonstructural components located in the upper levels of a structure experience higher lateral loads than components located in the lower levels, thereby justifying design equations incorporating a linear distribution of acceleration along the height of the building. Other nonlinear dynamic analyses performed on coupled primary-secondary systems (Lin and Mahin 1985, Toro et al. 1989, Sewell et al. 1989, Igusa 1990, Singh 1983) confirmed the important effect of the nonlinear behavior of the main structure or secondary system on the seismic forces transmitted to the secondary systems.

Freeman and Kehoe (1997) reviewed the data providing the basis for the linear distribution of accelerations over the height of the building that is used for computing seismic design forces in the 1994 NEHRP provisions for nonstructural components (ATC 1994) and in the 1994 edition of the Uniform Building Code (ICBO 1994). Contrary to the conclusions obtained by Schroeder and Bachman (1994) and the supporting analytical evidence noted above, the results of analyses and building data indicated that the assumption of a linear distribution of acceleration is not justified, and that over most of the height of the building, the floor accelerations are close to being constant due to contributions of higher modes. Therefore the use of code design force equations assuming constant acceleration should be preferred.

A similar study that incorporated the recorded acceleration data from the 1994 Northridge earthquake (Soong et al. 1997) showed that the data supported the design equation of the 1994 NEHRP provisions that predicts floor accelerations as a function of normalized building height above grade. The results of the study indicated also that in-structure accelerations do not appear to be reduced in longer period buildings, thereby suggesting that design forces for nonstructural components may be independent of structural period.

Villaverde (1996, 1997a, 2000) proposed a response spectrum technique to determine the design lateral forces on nonstructural components attached to building structures. The proposed procedure uses simplifying assumptions on the dynamic theory of primary/secondary systems interaction and the available ductility of nonstructural components to determine the design forces on secondary systems. Code design spectra can be used as ground motion input to the primary structure. The procedure was validated by time-history dynamic analyses on primary/secondary system assemblies subjected to representative earthquake ground motions.

Marsantyo et al. (2000) performed shake table tests of nonstructural components mounted on a supported structure in order to determine experimentally the acceleration amplification of nonstructural components mounted on floors of buildings. Both building equipment and building content were considered with several different floor anchorages. It was found that the ratio of building period to equipment period and the inherent damping of the equipment are the key parameters governing acceleration amplification. Lightly damped equipment rigidly anchored to the building floors produced higher acceleration amplification than heavily damped and freestanding equipment. It was also shown than the horizontal forces measured on equipment items exceeded the design seismic forces of the Uniform Building Code (ICBO 1997) for almost all cases. Finally, the use of a base-isolation system at the base of the equipment was very efficient in reducing the acceleration amplification.

Following the 1994 Northridge earthquake, a study was performed on combining the force levels used in the design of equipment anchorage and bracing with reconnaissance observations (URS Consultants/John A. Blume & Associates 1996). In a study by Naeim and Lobo (1998), data collected during the 1994 Northridge earthquake from instrumented buildings were used to assess the demand imposed by the seismic loading on nonstructural elements. Recorded force and displacement levels from six buildings ranging in height from 3 to 20 stories were compared to code requirements at the time of design as well as the latest code requirements. It was found that seismic force demands at the roof and base of several of the buildings considered were larger than prescribed by current codes. In certain buildings a good correlation between floor accelerations and nonstructural damage existed, while in others, extensive damage to nonstructural elements was inconsistent with lower levels of shaking. As noted by the authors, damage to nonstructural elements may be more dependent on detail and workmanship than on the actual level of strong shaking.

In a recent study by Rodriguez et al. (2000), a method to derive design horizontal forces is proposed. This method assumes that ductility affects only the floor accelerations associated with the first mode of the structure. Reasonable agreement was found with time-history analyses.

A comparison between experimentally measured seismic design forces and design code forces on nonstructural components is reported in a study by Marsantyo et al. (1998). The study considered both a rigidly fixed structure and a base-isolated one. It was found that when the natural period of the structure is close to the predominant period of the ground shaking, the dynamic amplification of mounted nonstructural elements is very important. It was also found that both the Japanese code (BCJ 1997) and the UBC code (ICBO 1997) underestimate the force demands on mounted nonstructural elements, especially when the damping of the main structure is low. When the structure was isolated at the base, the response of both the structure and the nonstructural elements was greatly reduced. The seismic forces on the nonstructural elements were well below the code requirements for the case of the isolated structure for all ground motions considered.

## 5.2 PAST STUDIES ON BUILDING CONTENTS

Rihal (1994) developed a test method for investigating the in-plane and out-of-plane seismic behavior of cantilever library shelving. Rihal's test method can be used to investigate the overall seismic behavior of library shelving items and to assess the seismic provisions of current regulatory codes.

An experimental evaluation of the seismic performance of modular office furniture systems was carried out by Filiatrault (1991). Five recorded ground motions, typical of the west coast of Canada and of the San Francisco Bay Area were used for testing furniture placed parallel and at

45° with the direction of the uniaxial ground shaking. It was found that the structural integrity of the system was not compromised during any of the earthquake ground motions considered, although a larger torsional response was observed when the furniture was oriented parallel to the ground shaking. Books located on the shelves of the unit toppled and could have caused injuries to occupants.

As part of a program to assess the seismic vulnerability of nonstructural components and building contents by the Public Works and Government Services Canada (PWGSC 1995), a set of 49 shake table tests were carried out (White 1999). The tests, which included horizontal and vertical input motions, showed that equipment restrained with properly designed methods performed very well, while unrestricted equipment suffered extensive damage. Equipment placed on isolation platforms also performed very well.

## 5.3 PAST STUDIES ON BUILDING SERVICE EQUIPMENT

### *5.3.1 Past Studies on Elevator Systems*

Research on elevator systems in the United States started only in the mid-1980s, and has focused primarily on the response of rails and counterweight systems, which are considered the most vulnerable components of elevators.

Yang et al. (1983) carried out the first published study on the seismic response of elevator systems. They constructed an experimental model of an elevator and counterweight system and tested it in the horizontal direction on a shake table under harmonic base excitation. A numerical model of the system was also developed for nonlinear time-history analysis. Only qualitative agreements could be obtained between the experimental and numerical results.

Tzou and Schiff (1987, 1988) studied numerically the in-plane impact problem between the counterweight and the rail and roller guides. The numerical model used was very similar to the one developed by Yang et al. (1983). The numerical results obtained indicated that current design practices underestimate the impact loading between the counterweight and the rail and roller guides. Tzou and Schiff suggested also that this impact loading could be reduced substantially by connecting the two rails together with a U-shaped tie rod.

Tzou and Schiff (1984, 1989) studied also numerically the seismic response of two modified counterweight configuration. It was shown that the introduction of a large gap between the weights and the frame of the counterweight was beneficial provided that the weights did not impact on the frame. The second configuration incorporated viscoelastic dampers between the weights and the frame of the counterweight. Again, the numerical results showed that the dampers could reduce the impact phenomenon but the overall dynamic response of the system could increase depending on the properties of the dampers.

Rutenberg et al. (1996), Segal et al. (1994, 1995, 1996) and Levy et al. (2000) investigated numerically the seismic response of counterweight systems under operating conditions (i.e., the counterweight moving vertically at a constant speed). The results obtained indicated that the seismic design loads prescribed by the ASME A17.A Code (American Society of Mechanical Engineers 1996) could be underestimated by as much as 500% to 650% for low-rise buildings and 50% to 250% for tall buildings located in a seismic zone 4 in the United States.

Following the 1994 Northridge earthquake, a study was performed to correlate structural analysis of elevator systems with reconnaissance observations (Finley et al. 1996).

Suarez and Singh (1996, 1998) developed an equivalent floor response spectrum technique to calculate the seismic response of rail-counterweight systems. The results showed that when the fundamental frequency of the rail-counterweight system is close to the building's fundamental frequency, very large displacements are induced. The helicoidal springs and rubber tires of the roller guides, however, would limit these displacements. It was also found that the position of the counterweight along the height of a building has a strong influence on the dynamic response.

Kelly and Tsai (1985) investigated experimentally, using a shake table, the seismic response of three light equipment items mounted on the top level of a 1/3-scale five-story frame. The shake table tests were conducted for the main structure with a fixed base and with the main structure isolated with rubber or lead rubber isolation bearings. It was found that tuning the isolation period of the main structure could control the seismic forces on the equipment items.

Juhn et al. (1990) conducted shake table tests of a 1/4-scale three-story steel frame incorporating a secondary system in the form of an inverted pendulum attached to the second story of the frame. The mass of the frame was ten times the mass of the secondary system. Experimental data on the dynamic primary-secondary system interaction were obtained and used to validate an analytical procedure proposed by the authors to construct floor response spectra.

### *5.3.2 Past Studies on Mechanical, Electrical, and Appliance Equipment*

Ohtani et al. (1992) conducted a comprehensive series of shake table tests on large-scale models of critical equipment in power plants. These tests were part of a larger program that had for objectives the evaluation of the seismic integrity of critical equipment in nuclear power plants and the validation of their seismic design methods.

A study by Morz and Soong (1997) presented an assessment of fire hazards related to the failure of water heaters in past earthquakes, as well as a number of proposed methods to mitigate this risk. The study focuses on different restraint systems and provides results from analyses, numerical simulations and shake table tests. The use of shut-off valves for fire mitigation is also considered in the study.

A study by Lam et al. (1987) investigates analytically and experimentally the seismic response of an air handling equipment unit mounted on vibration isolators. Electrometric isolators and uni-directional restraint isolators were tested. The observed experimental stiffness and damping characteristics of the isolators were used to develop analytical models able to accurately predict the seismic response of isolated units.

### *5.3.3 Past Studies on Ductwork and Piping Systems*

Nims and Kelly (1990) conducted shake table tests of three- and four-story steel frames incorporating a piping system representative of nuclear power plants. The interaction between the piping system and the frames was characterized, and three different restraining devices for the piping system were studied: snubbers, seismic stops, and energy-dissipating restraint devices.

Chiba et al. (1992) performed shake table tests of a three-dimensional piping system supported by a combination of fixed restraints and elastomeric isolation bearings. The objectives of the tests were to determine the dynamic response of a cracked pipe supported on elastomeric bearings and to quantify the effect of the pipe support stiffness on the crack growth.

Following the 1994 Northridge earthquake, a study was performed to combine structural analysis of elevator systems with reconnaissance observations (Ayres et al. 1996).

In a study by Tsuruta and Kojima (1988), an HVAC system was studied for the purpose of establishing a low-rigidity duct and support system as a seismic design procedure. Through experimental vibration tests and numerical modeling, it was concluded that the proposed system can be successfully applied in power plants as a cost-effective alternative to the more common high-rigidity systems.

## 5.4 PAST STUDIES ON BUILDING SERVICE EQUIPMENT

### *5.4.1 Past Studies on Communication Systems*

In 1982 in Japan, a field observation system was installed to monitor the seismic response of telecommunication equipment in a five-story telephone office building (Hiramatsu et al. 1988).

Sixteen accelerometers were installed on the equipment at various floors of the building, two accelerometers were installed on the roof steel tower, and one accelerometer was installed on an antenna above the tower. The system has been activated in several moderate earthquakes.

### *5.4.2 Past Studies on Computer Equipment*

In a study by Meyer et al. (1998), the performance of a restraint technique for mainframe computers and related equipment was investigated. The method, which simply makes use of tie-down rods connecting the equipment directly to the main concrete slab through the raised floor, was shown to be effective in keeping all attached equipment functional and undamaged under ground motions of the 1994 Northridge earthquake.

A study by Zhu and Soong (1998) examined the dynamics of freestanding bodies under combined horizontal and vertical ground shaking and provided a quantitative tool to assess the toppling risk of unrestrained equipment. It was found by the authors that the parameters influencing the toppling risk are the base excitation characteristics, the location of the excitation within the structure, the geometry of the equipment, and the properties of the surface at the interface of the equipment and the supporting base. It was also noted that the risk of toppling reduces with increasing size but is not dependent on the aspect ratio for very high levels of base excitation.

Another study (Amick et al. 1998) examined an isolation technique to protect semiconductor production facilities. As stated by the authors, the total cost of typical plants exceeds $1 billion of which more than three fourths represents the cost of the equipment. This equipment is extremely sensitive to vibrations, and is usually rigidly attached to the structure, which makes it prone to high accelerations during earthquakes. The isolation technique aims at keeping all equipment functional following a major earthquake by uncoupling the sensitive equipment from the rest of the structure.

Shake table tests were carried out on desktop computers mounted parallel and at 45° on a desk (Jin and Astaneh-Asl 1998). Two identical series of tests were carried out to assess the effect of installing connectors between the computers and the table. In the first series, where the computers were unattached, severe shaking representing a large seismic event resulted in the equipment being thrown off the table. In the second series, where the connectors were used, the computers remained in their initial position and were functional after the shaking despite the accelerations that reached peak values reaching as high as 6g to 8g for the computer and monitor, respectively.

## 5.5 PAST STUDIES ON INTERIOR ARCHITECTURAL COMPONENTS

When nonstructural elements, such as partitions, are not allowed to freely deform under the effect of an earthquake, they can influence the building response. In a study by Freeman (1977), results from an experimental evaluation on the racking response of building partitions is presented along with a method for determining the effect of partition stiffness and damping characteristics on the response of buildings.

The testing of the seven-story full-scale reinforced concrete building in Japan as part of a U.S.-Japan joint research program (U.S./Japan Joint Technical Coordinating Committee 1984) included nonstructural components. The following observations were made on the performance of interior partitions at different levels of roof drift:

- At a roof drift level of 1/1000: cracks occurred around the door openings of the partitions in the fifth story.
- At a roof drift level of 1/500: several doors could not be opened and extensive cracking of walls occurred especially around door openings.
- At a roof drift level of 1/250: many doors lost their function, mortar and plaster finishes began to fall off, and door jambs were separated from the partition.

- At a roof drift level of 1/125: boards were separated from furring frames, perpendicular partitions were severely damaged and separated.
- At a roof drift level of 1/60: doors at the third level showed out-of-plane buckling.

An experimental investigation was carried out by Rihal (1982) to determine the seismic performance of interior building partitions. The results of the static cyclic tests showed that the onset of cracking occurred at interstory drift levels of 0.07% to 0.26%. At interstory drift levels of 0.39% permanent damage was observed, while failure occurred at interstory drift levels of 0.52%. The amount of energy absorbed by the system was found to increase with increasing amplitude of loading, and was highly dependent on connection details and partition configuration.

A study on the dynamic characteristics of a long-span floor system revealed that the installation of internal partitions on the floor of a two-story gymnasium, part of a three-story school building increased the fundamental frequency of the floor system by 3%, while increasing the higher mode frequencies by 23% (Pernica, 1987). The presence of a full-span partition acted like a support and altered the vibration modes of the floor system.

## 5.6 PAST STUDIES ON EXTERIOR CONSTRUCTION

### *5.6.1 Past Studies on Exterior Curtain Walls*

Precast concrete panels have been commonly used over the past 20 years in the U.S. as exterior curtain walls for buildings. The anchors connecting the panels to the structural frames, which have been designed according to earlier code requirements, must be assessed and brought up to current code requirements. A study by Nielsen et al. (1998) examined the implementation of a retrofit strategy that consists of providing slotted holes to accommodate displacements between the panels and the main structure, thus limiting the level of seismic forces. It was shown experimentally that such a retrofit technique can successfully bring existing anchor systems to code displacements and force requirements. Nonetheless, the displacement capacity is also provided by the nonlinear deformation of the anchors, and typically, the pullout capacity is at least four times larger then the design requirement.

The performance of external curtain wall glazing subjected to seismically induced racking was experimentally investigated by King and Lim (1991). It was found that all systems tested were able to develop greater interstory drifts than expected. The authors concluded that full-scale laboratory testing could effectively be used to evaluate the performance of such elements, that loading rate had a significant effect on the performance, and that the number of loading cycles did not affect the behavior of the system significantly.

### *5.6.2 Past Studies on Appendages*

In the framework of the last phase of the U.S.-Japan Cooperative Research Program's six-story full-scale steel structure tests, the performance of cladding attached to the building was experimentally investigated (Wang 1987 and Roeder et al. 1987). Although the performance of the nonstructural elements was generally as expected, many unexpected failures occurred during the test. The inclusion of all nonstructural elements significantly influenced the initial lateral stiffness of the building. This increased stiffness rapidly deteriorated under cyclic action. It was found that both in-plane and out-of-plane drifts affect the performance of cladding. Stiffness, strength, and ductility characteristics of the connections were important for both directions of loading. A series of erection errors were observed during the test, and failures were attributed to poor workmanship in some cases. This emphasized the need for enhanced installation inspection and for simpler systems where both installation and inspection are straightforward. The geometric shape also played an important role in the performance of cladding. The L-shaped corner panels generally preferred by architects performed poorly. Connection flexibility and ductility, which is generally empirically assessed, proved to be an important parameter. Stiffer connection details caused premature cracking in concrete panels. The poor deformation compatibility between the wall panels connected at the beam level and the column covers connected at the columns led to excessive joint widths in the case where the panels were separating and to high stresses on the panel when the joint was closed. Among the different details considered, long ductile rods were able to accommodate large interstory drifts, whereas sliding connections encountered problems when insufficient slot lengths were provided. The latter are more sensitive to weathering and aging of the connection, improper installation, or poor detailing. It was further found that wide enough joints must be provided to avoid contact of panels during seismic loading, and that adjacent panels should be positioned in order to deform in a similar way under lateral loading.

Using an electromagnetic exciter as well as a wire-cutting technique, Nishizaka et al. (1996) studied the effect of nonstructural components on the natural period and damping characteristics of a real full-scale pre-fabricated steel buildings. The authors report that when all nonstructural components were installed, the natural frequency of the building shifted from 2.4 Hz to 6.1 Hz.

When the amplitude of the vibrations was increased, the stiffening effect was reduced because of damage to the nonstructural elements. The damping ratio also increased significantly when the nonstructural elements were added. The eigenproperties of the structure were also found to change significantly with the introduction of nonstructural elements.

### *5.6.3 Past Studies on Exterior Veneers*

Wang (1987) conducted static, free-vibration, and forced-vibration tests of a full-scale six-story reinforced concrete frame incorporating cladding systems made of precast concrete and fiber-reinforced glass panels.

Rihal (1988) performed in-plane cyclic racking tests on a solid precast concrete panel incorporating bearing connections at the bottom and threaded-rod lateral connections at the top. From the test results, the in-plane strength and deformation capacity of the panel were obtained.

Fischer et al. (2001) performed shake table tests on a full-scale two-story wood-frame house model. The structure was tested both with and without interior and exterior wall finish materials. The wall finish materials consisted of interior gypsum wallboard and exterior stucco. The 12-mm-thick gypsum wallboard panels were installed on all interior wall and ceiling surfaces. All surfaces were taped, mudded, and painted. The panels were oriented horizontally on the walls and fastened with 32-mm-long drywall screws spaced at 400 mm along the vertical studs. The ceiling panels were fastened with the same screws spaced at 300 mm on center. The exterior stucco finish was applied in three coats for a total thickness of 22 mm. The stucco was attached to the wood framing by a galvanized 17-gage steel wire lath fastened to the oriented strand board sheathing and vertical studs by 20-mm-long staples. The wall finish materials had a major influence on the resulting seismic response of the test structure. The test structure incorporating wall finish materials exhibited a nearly linear response, with a lateral stiffness much higher than the bare wood structure over all testing levels. Consequently, the wall finish materials reduced dramatically the seismic response of the test structure compared to the corresponding response of the bare wood structure. Also, there was a significant redistribution of anchor bolt forces in the structure incorporating wall finish materials.

### *5.6.4 Past Studies on Glass Doors, Windows, and Glazing*

Craig and Goodno (1981) performed laboratory tests of a window in a complete full-scale glass cladding panel in order to determine its natural frequencies, mode shapes, and modal damping ratios.

Researchers at the University of Utah and at Penn State University (Pantelides and Behr 1994, Behr et al. 1995) performed in-plane and out-of-plane dynamic tests of a section of a dry-glazed curtain wall, containing three glass panels and a wide mullion. Three types of glass having different thicknesses were tested. Included in the tests were annealed, heat-strengthened, and fully tempered glass in monolithic and laminated configurations.

In a study by Behr and Worrell (1998), the results of laboratory tests carried out over five years on the performance of various types of architectural glass and glazing systems under simulated earthquake loading are reported. Distinct dynamic drift levels were identified for glass cracking and glass fallout for the various types of glass systems. Considerable differences were found between the performance of different systems, notably that:

1. Laminated glass systems exhibited higher resistance to glass fallout than monolithic and filmed glass systems;
2. Annealed monolithic glass with unanchored film was not totally effective in preventing glass fallout;
3. Stiffer aluminum frames were less tolerant of glass-to-aluminum collision than more flexible frames, and were associated with glass fallout at lower drift levels; and
4. Glazing details were found to significantly affect the performance of architectural glass.

In a study by Memari et al. (2000), a model for predicting the ultimate drift capacity of full-size curtain-walls containing architectural glass based on the failure load of small-size glass was proposed. This approach, provides a cost-effective alternative to the full-scale testing recommended by the American Architectural Manufacturers Association to demonstrate acceptable performance of a glass wall system.

The performance of windows during the testing of a full-scale reinforced concrete building as part of a U.S.-Japan joint research program (U.S./Japan Joint Technical Coordinating Committee 1984) can be summarized as follows:

- No breakage of sliding window glass occurred;
- Cracks were observed in fixed windows with hardening putty when the story drift was around 1/1,500-1/500;
- Cracks were found in fixed windows with elastic sealant when the story drift was around 1/125-1/73; and

- No fragments of broken glass fell in cases of glass with polyester films or wired glass.

## 5.7 PAST STUDIES ON GLOBAL PERFORMANCE OF NONSTRUCTURAL ELEMENTS

The expected performance of nonstructural elements of a 27-story building located in Los Angeles, California (Shipp and Johnson, 1990) was assessed using calculated interstory drifts and floor accelerations. The benefits of different selective strengthening measures were also evaluated. Considered in the study were the expected performance of exterior cladding, interior partitions, as well as ceilings and floors. The authors concluded that the expected damage to nonstructural elements was considerable, and that it could be reduced by:

- Increasing anchoring strength, particularly for equipment mounted on vibration isolators;
- Installing lateral bracing for suspended equipment, particularly for fan-coil units;
- Improving bracing requirements for units located above living units, where later access for repair may require partial demolition;
- Increasing bracing requirements for fire sprinkler piping;
- Improving attachment details for precast panels to accommodate structural displacements; and
- Improving detailing of interior partitions to accommodate structural displacements.

The authors estimated that implementation of these measures would result in a reduction of the total losses during the earthquake from 18% to 13%, and estimated the cost of implementing these measures at 10% of the value of the reduced losses.

A comprehensive study was undertaken at UC Berkeley to estimate the potential seismic nonstructural losses common to classrooms, offices, libraries, and laboratories (Comerio, 2000, Comerio et al. 2001). A more detailed investigation was also carried out on five case study laboratory spaces. The work includes a literature review on possible measures to mitigate the potential nonstructural hazards and on cost estimates on the mitigation measures as well as estimates on potential loss reduction. An evaluation of the loss reduction measures as part of an integrated performance-based structural and nonstructural retrofit strategy is also presented.

The Bay Area Regional Preparedness Project (BAREPP 1990) discusses damage caused to nonstructural elements during the 1989 Loma Prieta earthquake and proposes methods for businesses to prepare for a subsequent earthquake. Included in this document is a videocassette on nonstructural hazards as well as an information guide.

The issue of life hazard caused by nonstructural elements was investigated by Elsesser (1984). Among the hazards identified are cladding, infill walls, glass, partitions, ceilings, stairs, elevators, equipment, shelving, and contents. A methodology for assessing the global life hazard from nonstructural elements is also proposed.

# 6 Computerized Database

An extensive amount of literature has been published over the past twenty years on the performance of nonstructural elements in earthquakes. A computerized database has been developed in this project to centralize this large amount of information as well as to facilitate any future literature searches by researchers, practicing engineers, manufacturers, or any other parties interested in the topic. This database includes earthquake reconnaissance reports, past research and many specific requirements published by different organizations, as well as manufacturers on the design of nonstructural elements that are usually not accessible by literature search engines. A total of over 400 documents are currently included. The database is accessible from the Internet at the following website: http://seismic.ucsd.edu/peer/nonstructural.html and includes an interactive search tool. To facilitate the insertion of the literature that has not yet been included, as well as the addition of future work on the topic, an interactive submission form is also available.

## 6.1 SEARCH ENGINE

To use the literature database search engine, one or more fields must be entered in the space indicated by 1 for a search by author name and by 2 for a search by words in the title (see Fig. 6.1). Once the fields have been entered, the Search button, indicated by 3 on Fig. 6.1, is pressed to activate the search. The Clear button, indicated by 4 on Fig. 1 erases all fields that have been entered.

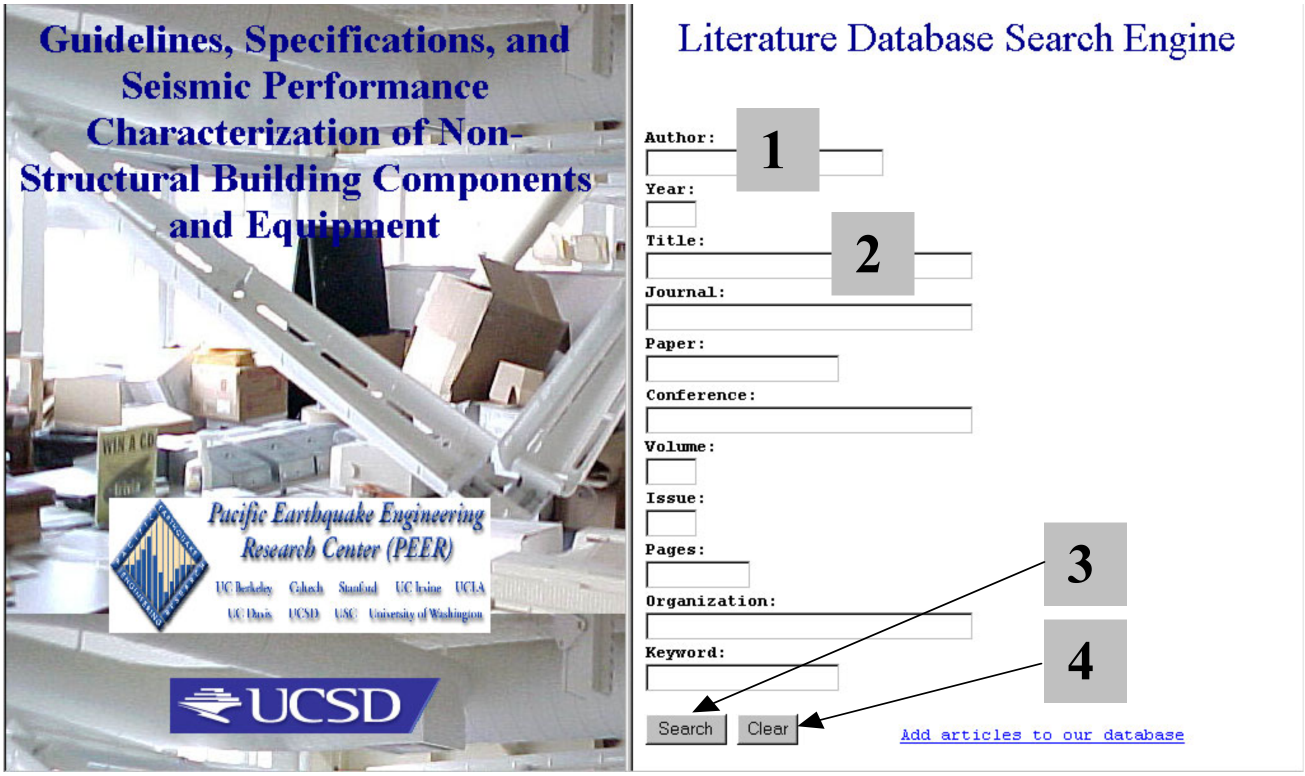



**Figure 6.1 Searching the Database**

To illustrate a search by author, the name Merz is entered in the author space (marked by 1 on Fig. 6.1). To activate the search engine, the Search button is pressed (2 on Fig. 6.1) and the results are displayed on the screen (Fig. 6.2).


**Figure 6.2 Results from Search by Author**

To illustrate a search by words in the title, "cladding components" is entered in the title space (indicated by 2 on Fig. 6.1). To activate the search engine, the Search button is pressed (3 on Fig.6.1) and the results are displayed on the screen (Fig. 6.3).

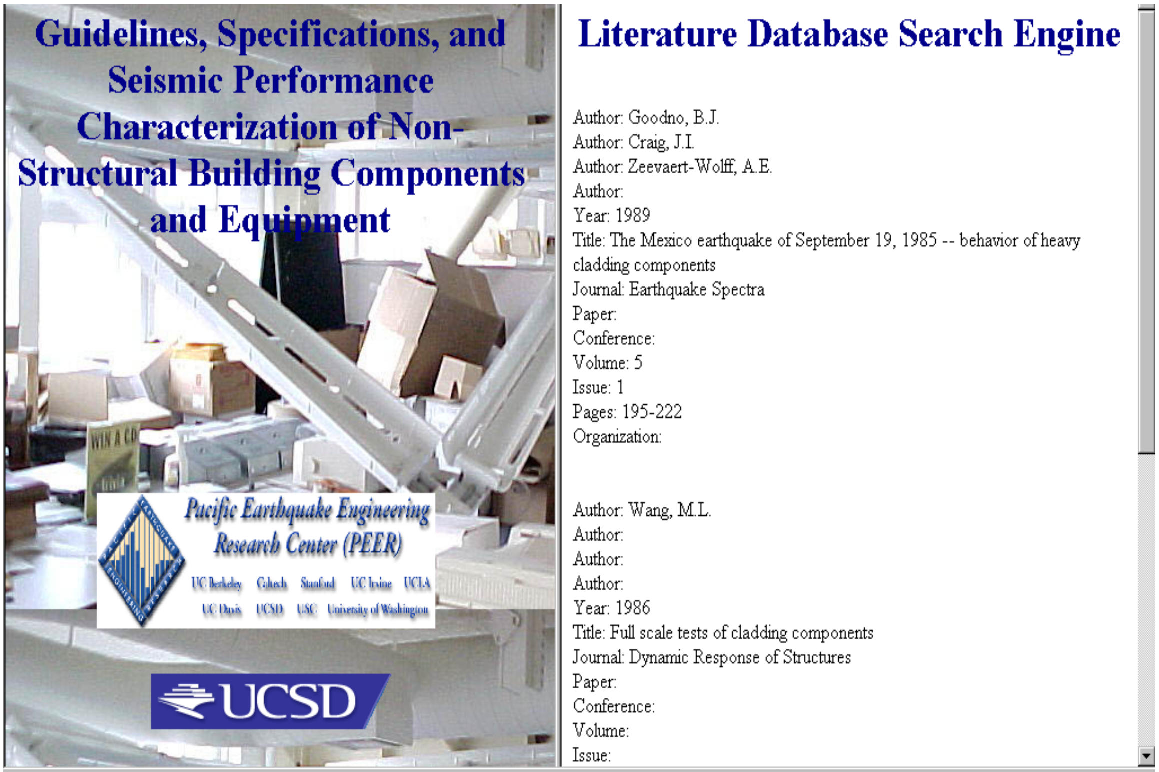
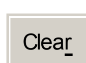



**Figure 6.3 Results from Search by Words in Title**

## 6.2 LITERATURE SUBMISSION ENGINE

To submit a reference to be added in the database, press the Add references to our database link located to the right of the 

button on the main page (see Fig. 6.1). All the available information is then entered in the Article Addition Form (Fig. 6.4).


**Figure 6.4 Article Addition Form**

Once completed, the Add Record button located at the bottom of the page is pressed. A confirmation page summarizing the information provided appears. The article will then be reviewed for pertinence to the topic and then added if the information provided is sufficient to form a complete reference.

# 7 Summary of Gaps in Knowledge and Recommendations for the Development of a Rational Research Plan on Nonstructural Building Components

## 7.1 GENERAL RECOMMENDATIONS

### *7.1.1 Development of Efficient Data Collection Methods*

The statistical summaries from past earthquakes on the damage to nonstructural building components play a very important role in formulating code design/retrofit improvements. However, there are problems associated with the collection of information. The most significant problem is the lack of record of relevant engineering details associated with the failure of nonstructural building components. This inadequate reporting of information, both following the main shock and potential strong aftershocks, often makes it difficult to establish the real cause of the failures. Non-engineering personnel, lacking the training required to assess damage and identify modes of failure, usually do the task of collecting information related to the failure of nonstructural building components.

There is an urgent need for the development of a post-earthquake field reconnaissance “tool kit” for nonstructural components. This tool kit could take several forms and should include sets of sample forms to efficiently collect information on the damage and failure of nonstructural building components in the aftermath of an earthquake.

Furthermore, there is an urgent need to establish an overall data collection strategy. As stated by Reitherman (1998), a significant effort is necessary to yield reliable and useful data. Elaborating such a strategy would require an efficient distribution of the efforts between (1) questionnaires for inexpensive widespread data collection by non-qualified personnel and (2) guidelines for collecting higher quality and consequently more expensive information by qualified personnel. A subsequent effort to coordinate both sets of data and to correlate them to ground shaking intensity and structural performance is also needed. This strategy must be pre-established and "ready to go" because a significant portion of the damage to nonstructural elements is cleaned up and rapidly repaired following an earthquake to allow occupants to resume their activities. The information must be gathered within the first three days following an earthquake. A comprehensive data collection effort may require as much as 12 to 24 months after the earthquake to be completed, and entails significant interaction among all concerned parties, including research-oriented organizations and universities, practitioners, industry groups, standards organizations, political organizations, as well as government agencies.

### *7.1.2 Development of Post-Earthquake Nonstructural Inspection Procedures*

Based mainly on observations from the Northridge earthquake, there has been an inconsistency in the application of post-earthquake safety criteria to nonstructural components compared to the application of similar safety criteria to structural components and systems. For example, at least 75 hospitals tagged green following the Northridge earthquake suffered sufficient nonstructural damage to prevent operation of many services and closure of certain areas of the buildings (OSHPD 1994). Serious consequences could have resulted had a strong aftershock hit these facilities and caused further damage to nonstructural components.

Specific procedures for the post-earthquake safety inspection of nonstructural components need to be developed. These procedures will need to be harmonized with the current structural safety evaluation procedures.

### *7.1.3 Development of Seismic Analysis Methods for Nonstructural Components*

The determination of seismic design forces on nonstructural components has become more complicated due to the variety of procedures available, the various assumptions that these procedures are based on, and the difficulty of classifying the variety of components in a rational way. The complexity of many of these procedures, however, has precluded their utilization in building code provisions. Furthermore, current procedures are based mainly on judgment and intuition rather than engineering research results. One of the main problems associated with the evaluation of seismic design forces is the difficulty in classifying the various types of nonstructural systems and components. Historically, nonstructural components have been classified based on their usage characteristics. Another classification of nonstructural components based perhaps on their seismic response characteristics needs to be elaborated.

Another urgent need is the evaluation of current seismic design procedures and the development of improved yet, sufficiently simple, methods (static and dynamic) for determining realistic seismic design forces and drifts on floor-mounted nonstructural components. These methods need to address the following aspects that have not been fully considered to date:

- The influence of damping properties of structural and nonstructural components
- The influence of structural and nonstructural components non-linearity
- The influence of structural torsion
- The distribution of floor accelerations along the building height
- The seismic response of base-isolated or passively controlled secondary systems
- The interaction between structural and nonstructural elements and interconnected nonstructural elements

Data recorded from instrumented buildings during recent earthquakes in the United States and Japan should be used to assess the validity and effectiveness of simplified methods of analysis included in current design methods. Further instrumentation of buildings to provide information specific to the response of nonstructural components during future earthquakes is also needed.

### *7.1.4 Development of Experimental Seismic Qualification Procedures for Nonstructural Components*

An extensive experimental program is urgently needed to complement the substantial amount of ongoing analytical studies on the seismic behavior of nonstructural components. The development of static and dynamic (shake table) test protocols is required to characterize the physical properties of nonstructural components and qualify both their structural and functional performances during seismic events.

### *7.1.5 Application of Performance-Based Earthquake Engineering to Nonstructural Components*

With the development of performance-based earthquake engineering, harmonization of the performance levels between structural and nonstructural components is necessary. Even if the structural components of a building reach an immediate-occupancy performance level during a seismic event, equipment failure inside the building can lower the performance level of the entire building system. This reduction in performance caused by the vulnerability of nonstructural components has been particularly evident in several buildings during the recent 2001 Nisqually earthquake in the Seattle-Tacoma area (Filiatrault et al. 2001), as already noted in Chapter 1.

As a result of improved building codes, structural systems are expected to perform better in a seismic event. Increasing the strength of buildings results in structures responding more in the elastic range, thereby increasing the floor acceleration levels. Elaborate detailing of structural systems to provide a stable post-yielding behavior, without degradation, also contributes to increasing the effective accelerations soliciting the nonstructural components, as the initial stiffness is recovered at every cycle. Furthermore, as illustrated by the 2001 Nisqually earthquake, the response of the structural system to lower seismic events may be linear while significant accelerations are experienced in the building. This underscores the importance of linking design requirements for nonstructural elements to the response of the structural system under various ground shaking intensity levels. Also, a sensitivity study on the effect of the main structural system, i.e., stiff wall systems versus flexible frame systems, on the expected damage to both acceleration and displacement sensitive nonstructural elements should be available to structural engineers in order to fully assess the cost-benefit implications of design decisions. Holmes (1984) suggested an approach that required identifying the structural response parameters and their relationship to nonstructural damage. Various building deformation modes such as shear-type distortions typical of framed structures, bending-type distortions typical of wall structures, as well as soft-story mechanisms are considered.

Methods that have been developed for the application of performance-based earthquake engineering should be considered and tailored to nonstructural components. These methods should make use of the analysis tools to be developed as suggested in 7.1.3, the experimental data obtained from the seismic qualification protocol to be developed as proposed in 7.1.4, and should also include cost-benefit decision tools in order to select appropriate performance levels for nonstructural components. These performance levels for nonstructural components also need need to be harmonized with the structural performance levels.

Physical installation methods consistent with these performance levels need to be developed. These installation methods must also consider the interaction of different nonstructural components when installed side by side. In many instances, it has been observed that the performance of adjacent nonstructural components has been lowered because of a lack of coordination in installation procedures causing the total system to function at a lower performance level than intended (Reitherman and Sabol 1995).

### *7.1.6 Comprehensive Assessment and Design of Nonstructural Elements*

There is an urgent need to create a comprehensive framework for the seismic assessment and design of nonstructural elements. An important amount of specific information is available through field reconnaissance reports, published research, design codes, and specific requirements, but very little has been done to incorporate this into a complete procedure that fully accounts for the complex interaction between the main structural system and the nonstructural elements. The poor performance of nonstructural elements in past earthquakes triggered changes to code requirements resulting in reassessment of design force requirements or detailing to provide displacement capacity. Although these enhanced requirements led to better performance of the specific elements considered, a general methodology is needed that is consistent with the current performance-based seismic design philosophy. The performance of buildings can no longer be assessed independently of the performance of the nonstructural elements and vice versa. It is therefore sensible to define both the structural and nonstructural elements early in the design phase, and to explicitly consider them jointly during the subsequent design iteration process.

Figure 7.1 illustrates a possible comprehensive framework for the seismic assessment and design of nonstructural elements. Once a performance level is set for a given ground motion intensity, the first task in this procedure is to clearly identify all nonstructural elements that are considered in the design process. A clear evaluation of the load path from the center of mass of each nonstructural element to the main structure must be determined, and all the connectors allowing this load path must be identified and characterized. The connection points of each nonstructural element to the main structure and to other nonstructural elements must be identified. This process allows to locate the points where the demand on the nonstructural elements must be evaluated. Among these elements, those affecting the response of the main structural system must be identified and their stiffness, strength and damping characteristics must be included in the structural response of the main system.

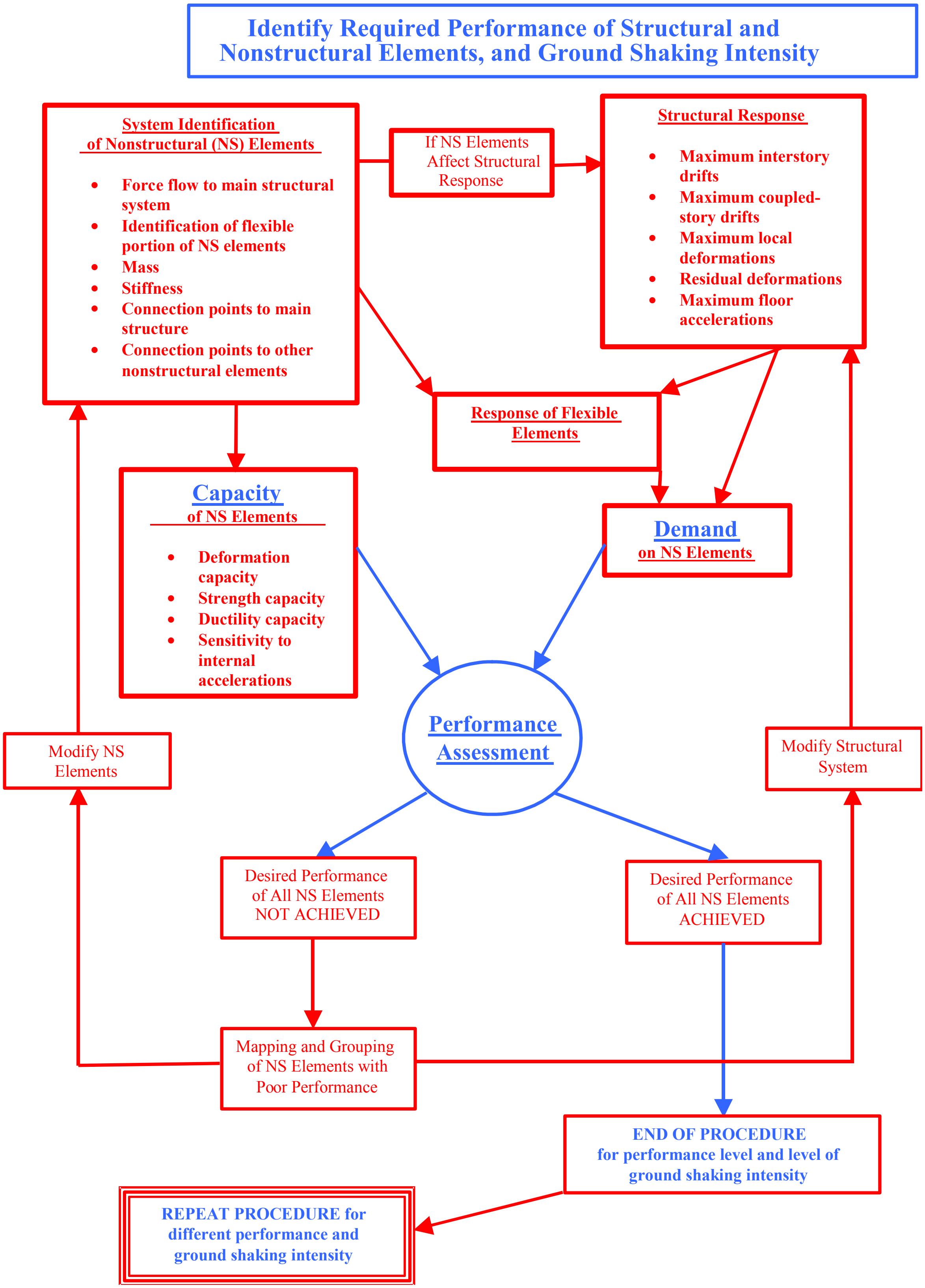



**Figure 7.1 Framework for the Seismic Assessment and Design of Nonstructural Elements**

The structural response must then be evaluated based on a required degree of modeling sophistication. In many cases, not considering the nonlinear characteristics of the structure will lead to unrealistic estimates of the structural response. The structural response must provide the following information:

- Maximum interstory drift
- Maximum coupled-story drifts (the required coupled drifts are defined by the boundary conditions or connection points of the nonstructural elements to the main structure, for example a piping system connected at the first and third floors)
- Maximum local deformations. In many cases, especially for multi-span frames with different span lengths, the local deformation at the location of plastic hinges may be significantly larger than the average interstory drift due to geometric considerations.
- Residual deformations
- Maximum floor accelerations

This list of response indices influencing the performance of nonstructural elements can be updated as more particular nonstructural elements are identified. At this point, the performance level of the main structural system is verified. Although this phase is not explicitly shown in Figure 7.1 for sake of clarity, it nevertheless remains a very important stage of the performance assessment framework. The structural system is modified until the required performance level is reached.

These structural responses are then fed as input to the dynamic evaluation of the flexible portions of the nonstructural elements. The results from this dynamic evaluation, which is also carried out at the required degree of sophistication, along with results coming directly from the response of the main structure, define the demand side of the performance assessment process.

Based on the nonstructural system identification process, and on the characteristics of each nonstructural element, the capacity of each nonstructural element is established. This process consists of coupling different values of an input quantity to the performance of the nonstructural element. This process is similar to the performance-based curve of a structural system, where the force-deflection relationships define different levels of performance. This must be carried out, when deemed applicable, to the following input quantities:

- For each rigidly attached nonstructural element, and for the rigidly attached portions of flexible nonstructural elements, as defined earlier, the strength and ductility characteristics of connectors must be identified.
- For each nonstructural element attached to more than one point to the main structure or to other nonstructural elements, a thorough survey of the displacement capacity between these attachment points must also be carried out.
- For each flexible portion of nonstructural elements, the force deflection characteristics must be assessed and coupled with performance levels.
- For internally sensitive equipment, the performance for various acceleration levels must be determined.

The determination of the capacity can be as simple as acceptable or defined by more complex functions defining different levels of performance. The performance assessment process then becomes the coupling between the demand and the capacity, as previously defined. Each component is first assessed individually, and then all performance levels are summed up to determine a global performance. The nonstructural elements that did not perform to the required level are then identified and fed as input to the mapping and grouping process. This process is very important to determine if there is a location in the building where the nonstructural elements are performing poorly, and to identify families of nonstructural elements performing poorly. At this point the designer must loop over the design process. Based on the mapping and grouping information, a design decision must be made. The first possibility is to intervene at the level of the nonstructural element. A choice of changing either the component itself, modifying its boundary conditions, or introducing a new detail to the same system is possible. Another possibility can be to modify the structural system to reduce the demand side of the performance assessment. Hybrid solutions combining changes to both the nonstructural elements and to the structural system can also be considered.

This process is repeated until the required performance levels of both the structural system and all nonstructural elements are achieved. The process is also repeated for different couples of performance levels and ground shaking intensity.

It must be noted that a series of indications on the global characteristics of structures incorporating different lateral load-resisting systems would be useful beforehand as a guideline to designers. Guidelines such as inherently high drift or floor accelerations would allow choosing either the nonstructural system itself or initially modifying a detail to account for this characteristic structural response. It may even be assessed in the early stages of the design process that the structural system should be modified, either by changing the lateral load-resisting system or by isolating or bracing an existing system.

Also, considering that the performance of nonstructural elements in a building is usually not a function of building height, structural systems whose response quantities are constant with height are likely to lead to better overall performance of the nonstructural elements.

## 7.2 RECOMMENDATIONS FOR BUILDING CONTENTS

Past earthquakes have shown that damage to building contents is not correlated with the intensity of the shaking as usually measured. For example, many buildings located in the most intensely shaken areas affected by the 1994 Northridge earthquake suffered surprisingly low contents damage. Analytical studies on the effect of horizontal and vertical accelerations on freestanding content items could explain this phenomenon and should be undertaken.

Library shelving as well as storage rack systems have proven to be very vulnerable to ground shaking. Further work is needed to mitigate damage, which typically comes in the form of complete collapse, partial failure without collapse and/or content toppling. Although increasing the strength of these systems is necessary to prevent full collapse, further investigations on the correlation between rack response and material toppling is needed. Performance of these systems must be defined on a dual level, to encompass both the structural performance of the shelving systems as well as toppling of contents. As shown in recent full-scale shake table tests on warehouse storage racks (Filiatrault and Christopoulos 2001), these systems can typically undergo large inelastic displacements in the in-plane direction without compromising their vertical load-carrying capacity and while introducing a considerable amount of viscous damping. Detailing these systems to increase their ductility is likely to increase their global performance by limiting the accelerations on the contents. In the out-of-plane direction, where accelerations are expected to be larger as a result of the stiffer structural system, further practical recommendations on restraining contents are needed. Further research is also needed to define the critical loading combinations for different failure modes, especially for the shelving system anchor uplift.

## 7.3 RECOMMENDATIONS FOR BUILDING SERVICE EQUIPMENT

### *7.3.1 Recommendations for Elevator and Escalator Systems*

The practical implementation of seismic and displacement switches in elevator systems is not easy (Suarez and Singh 2000). One of the main problems is that these switches can be activated by non-seismic disturbances such as vibrating machinery, nearby traffic, or nearby construction work. The development of a new generation of seismic sensors for elevator systems is an area of active research that draws in researchers operating outside traditional earthquake engineering disciplines. Shake table testing of seismic and displacement elevator switches commercially available in the United States also needs to be undertaken.

Furthermore, results of the few dynamic analyses performed on elevator systems seem to indicate that the seismic design procedures included in the ASME A17.1 Code (American

Society of Mechanical Engineering 1996) may not be conservative for all cases. The development of rational analytical/numerical models to evaluate the demand and capacity of elevator systems is a worthwhile undertaking within the general framework of performance-based earthquake engineering.

### *7.3.2 Recommendations for Mechanical, Electrical, and Appliance Equipment*

The performance of mechanical and electrical systems in past earthquakes has been good for rigidly mounted systems provided that anchors were designed for code-level forces. However, some of the anchors that were well sized to carry these seismic forces failed prematurely as a result of anchor pullout. Better detailing of the connection between the anchor and the main structure (typically concrete slabs) is needed to insure that the forces are transmitted to the main structural system.

On the other hand, the poor performance of isolated equipment demonstrated the need to fully reassess the seismic performance of systems mounted on springs or rubber. A coordinated effort between the structural and mechanical engineering professions is needed to insure adequate performance of these vibrating systems under both low-amplitude, higher-frequency mechanical vibrations and higher-amplitude vibrations induced by seismic loading of the main structure. A further effort to realistically assess the amplification factors of existing equipment mounted on these isolation systems is also needed to guide strengthening work on existing equipment.

For water heaters, it is suggested that the restraints be engineered to assure adequate protection of these systems. Shut-off valves, triggered by ground shaking, have also been found to effectively reduce the risk of fire caused by the failure of water heaters, but they do not prevent damage to the water heater.

### *7.3.3 Recommendations for Ductwork and Piping Systems*

The 1994 Northridge earthquake demonstrated that the common belief that small-diameter piping (less than 1 in.) needs no bracing may not be true. The evaluation of current bracing requirements for piping systems, supported by systematic analytical and experimental studies, would be beneficial in order to provide adequate bracing requirements for the whole range of piping systems.

For fire sprinkler systems, the elaboration of flexible details at the connection of the main water branch and the smaller water branches located within the suspended ceiling and/or the uncoupling of the sprinkler system from the suspended ceiling would greatly enhance the performance of these systems during earthquakes.

## 7.4 RECOMMENDATIONS FOR BUILDING UTILIZATION EQUIPMENT

### *7.4.1 Recommendations for Computer Equipment*

Computer equipment is primarily susceptible to toppling or sliding during earthquakes, but can also be affected by interstory displacements of the main structure or of the raised floors. Techniques for restraining such equipment are available and have proved effective in past earthquakes. However, computer equipment is also susceptible to internal damage. As effective restraining strategies are implemented on computer equipment, the internal accelerations are likely to increase, and can cause failure of the sensitive equipment contained within. A coordination effort is necessary between structural and computer engineers to assure that the seismic protection is not internally detrimental to the system. A better understanding of floor accelerations as noted in Section 7.1.3, as well as proper communication of this information to computer engineers would greatly reduce the risk of failure of computer systems during earthquakes.

## 7.5 RECOMMENDATIONS FOR INTERIOR ARCHITECTURAL COMPONENTS

### *7.5.1 Recommendations for Ceiling Systems*

Past earthquakes demonstrated that the performance of suspended ceiling systems was greatly impaired by the interaction between the different structural and nonstructural interconnected elements. Further research is needed to define recommendations on either achieving deformation compatibility between the components or uncoupling these systems and allowing them to move independently. Furthermore, the effect of the spacing between the ceiling and the underside of the above floor on the amplification of the ceiling response should be investigated. The increased stiffness resulting from a reduced spacing is likely to reduce the maximum deformations of the system and therefore mitigate the problem of deformation compatibility with other systems.

## 7.6 RECOMMENDATIONS FOR NONSTRUCTURAL EXTERIOR CONSTRUCTION

### *7.6.1 Recommendations for Exterior Veneers*

Although the beneficial effect provided by the wall finish materials was very pronounced in the wood-frame house tested by Fischer et al. (2001), it is unclear if this effect would also occur in larger wood-frame structures with more complicated geometry. Nevertheless, the results of that study have provided a motivation to examine further the effectiveness of wall finish materials as potential structural components of lateral load-resisting systems of wood-frame structures. Several issues need to be addressed before these materials can be effectively considered in design. For example, the method of attachment of stucco to the wood framing should be researched in order to evaluate current practices, and possibly develop improved attachment methods that could mobilize the lateral stiffness and strength of stucco for the sequence of earthquakes that a wood-frame building could experience during its lifespan.

### *7.6.2 Recommendations for Glass Doors, Windows, and Glazing*

The performance of glass doors, windows and glazing during earthquakes is highly dependent on the deformation capacity provided to the brittle material with respect to its supporting frame. More elaborate specifications are needed to fully address this issue, for all types of glass and different pane sizes. Furthermore, in the framework of performance-based earthquake engineering, an effort to better correlate the expected interstory distortion of the main structural system to the displacement capacity of the glass systems is needed.

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