BIBLIOGRAPHIC INFORMATION

PB96-193800

Report Nos: NISTIR-5857

Title: State of the Art Report on Seismic Design Requirements for Nonstructural Building Components.

Date: Jun 96

Authors: L. T. Phan and A. W. Taylor.

Performing Organization: National Inst. of Standards and Technology (BFRL), Gaithersburg, MD. Structures Div.

NTIS Field/Group Codes: 89H (Building Equipment, Furnishings, & Maintenance), 89E (Building Standards & Codes), 89D (Structural Analyses)

Price: PC A06/MF A01

Availability: Available from the National Technical Information Service, Springfield, VA. 22161

Number of Pages: 78p

Keywords: *Earthquake engineering, *Building codes, *Lighting equipment, *Ceilings, *Sprinkler systems, Seismic design, United States, New Zealand, Japan, Seismic effect, Earthquake damage, Damage assessment, Dynamic response, Loads(Forces), Displacement, Requirements, Nonstructural components.

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PB96-193800

NIST-114
(REV. 6-93)
ADMAN 4.09

U.S. DEPARTMENT OF COMMERCE
NATIONAL INSTITUTE OF STANDARDS AND TECHNOLOGY

MANUSCRIPT REVIEW AND APPROVAL

ERB CONTROL NUMBER	DIVISION
W96-1070	861
PUBLICATION REPORT NUMBER	CATEGORY CODE
	140
PUBLICATION DATE	NUMBER PRINTED PAGES

INSTRUCTIONS: ATTACH ORIGINAL OF THIS FORM TO ONE (1) COPY OF MANUSCRIPT AND SEND TO THE SECRETARY, APPROPRIATE EDITORIAL REVIEW BOARD

TITLE AND SUBTITLE (CITE IN FULL)

State of the Art Report on Seismic Design Requirements for Nonstructural Building Components

CONTRACT OR GRANT NUMBER

TYPE OF REPORT AND/OR PERIOD COVERED
NISTIR

AUTHOR(S) (LAST NAME, FIRST INITIAL, SECOND INITIAL)
Phan, L.T. and Taylor, A.W.

PERFORMING ORGANIZATION (CHECK (X) ONE BOX)
- [X] NIST/GAITHERSBURG
- [ ] NIST/BOULDER
- [ ] JILA/BOULDER

LABORATORY AND DIVISION NAMES (FIRST NIST AUTHOR ONLY)
Building and Fire Research Laboratory, Structures Division (861)

SPONSORING ORGANIZATION NAME AND COMPLETE ADDRESS (STREET, CITY, STATE, ZIP)

NIST Category No.
NIST- 140

PROPOSED FOR NIST PUBLICATION
- [ ] JOURNAL OF RESEARCH (NIST JRES)
- [ ] J. PHYS. & CHEM. REF. DATA (JPCRD)
- [ ] HANDBOOK (NIST HB)
- [ ] SPECIAL PUBLICATION (NIST SP)
- [ ] TECHNICAL NOTE (NIST TN)
- [ ] MONOGRAPH (NIST MN)
- [ ] NATL. STD. REF. DATA SERIES (NIST NSRDS)
- [ ] FEDERAL INF. PROCESS. STDS. (NIST FIPS)
- [ ] LIST OF PUBLICATIONS (NIST LP)
- [X] NIST INTERAGENCY/INTERNAL REPORT (NISTIR)
- [ ] LETTER CIRCULAR
- [ ] BUILDING SCIENCE SERIES
- [ ] PRODUCT STANDARDS
- [ ] OTHER

PROPOSED FOR NON-NIST PUBLICATION (CITE FULLY) [ ] U.S. [ ] FOREIGN
NISTIR 5857

PUBLISHING MEDIUM
- [X] PAPER
- [ ] DISKETTE (SPECIFY)
- [ ] OTHER (SPECIFY)
- [ ] CD-ROM

SUPPLEMENTARY NOTES

ABSTRACT (A 2000-CHARACTER OR LESS FACTUAL SUMMARY OF MOST SIGNIFICANT INFORMATION. IF DOCUMENT INCLUDES A SIGNIFICANT BIBLIOGRAPHY OR LITERATURE SURVEY, CITE IT HERE. SPELL OUT ACRONYMS ON FIRST REFERENCE.) (CONTINUE ON SEPARATE PAGE, IF NECESSARY.)

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KEY WORDS (MAXIMUM OF 9; 28 CHARACTERS AND SPACES EACH; SEPARATE WITH SEMICOLONS; ALPHABETIC ORDER; CAPITALIZE ONLY PROPER NAMES)

building technology; building code; ceiling component; earthquake; nonstructural component; nonstructural damage; seismic design requirement

AVAILABILITY
- [X] UNLIMITED
- [ ] FOR OFFICIAL DISTRIBUTION - DO NOT RELEASE TO NTIS
- [ ] ORDER FROM SUPERINTENDENT OF DOCUMENTS, U.S. GPO, WASHINGTON, DC 20402
- [X] ORDER FROM NTIS, SPRINGFIELD, VA 22161

NOTE TO AUTHOR(S): IF YOU DO NOT WISH THIS MANUSCRIPT ANNOUNCED BEFORE PUBLICATION, PLEASE CHECK HERE. [ ]

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PB96-193800

NISTIR 5857

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June 1996

U.S. DEPARTMENT OF COMMERCE
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TECHNOLOGY ADMINISTRATION
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NATIONAL INSTITUTE OF STANDARDS
AND TECHNOLOGY
Arati Prabhakar, Director

Abstract

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1. INTRODUCTION

1.1 General

Nonstructural building components are elements within or attached to buildings to provide them with essential services and functions, such as heating and cooling, lighting, escalators, electrical power, etc. These components are not a part of the building structural system, and are not designed to contribute to the resistance of earthquake forces. In most building codes, nonstructural components are commonly grouped into two categories: (1) *architectural*, and (2) *mechanical and electrical*. *Architectural* nonstructural components include, for instance, cladding, suspended ceilings, exterior and interior nonbearing walls and partitions, parapets, penthouses, etc. *Mechanical and electrical* nonstructural components include most building secondary systems such as boilers, furnaces, storage tanks, HVAC systems, piping systems, elevator components, electrical systems, and lighting fixtures, etc.

In several past earthquakes, it has been documented that damage to both *architectural* and *mechanical* nonstructural building components can have a great effect on the safety of occupants, functionality of facilities, and loss of property. While statistical cost data for nonstructural damage are scarce, it is widely agreed and reported that the economic effects of all nonstructural damage combined generally exceed those of structural damage in an earthquake.[22,23] In many cases, these "indirect losses" due to damaged equipment, lost inventory and records, and revenue can be two to three times greater than the cost of replacing collapsed buildings or structures, as often reported in the 1971 San Fernando[1,2], the 1989 Loma Prieta[3], and the 1994 Northridge earthquakes[4].

Earthquake damage to nonstructural components has been a continuing concern. In fact, one of the early recommendations, made as a result of the *National Workshop on Building Practices for Disaster Mitigation*[5] which was held more than two decades ago, was:

> *A multi-disciplinary program of analytical, experimental, and design studies should be conducted to acquire knowledge and develop standards for improving practices of design for nonstructural building elements.*

More recently, in a 1995 report prepared by the Office of Technology Assessment for the U.S. Congress, entitled "*Reducing Earthquake Losses*"[22], it is stated, in addressing the future needs for reducing earthquake losses to the built environment, that "*it is time for new building seismic engineering research to consider the next problem: **reducing nonstructural and contents damage***".

Most of the research efforts in seismic engineering to date, however, have focused on improvements of the structural design of buildings to prevent total collapse. This is consistent with the life-safety philosophy inherent in the model building codes and justifiably so since the concern for total building collapse, which has more serious life safety implications, is naturally greater than the concern for local failure of nonstructural components. As a result, most newly constructed buildings stand a good chance that they will not collapse during an anticipated earthquake. It is only recently that more research attention has been paid to the performance of nonstructural components and secondary systems. The result is that many of the current model building codes and seismic provisions in use in the U.S., such as the 1994 Uniform Building Code[6], the 1994 Standard Building Code[7], and the 1994 NEHRP Recommended Provisions for Seismic Regulations for New Buildings[9,10], now contain revised and more stringent seismic design requirements for various nonstructural building components and equipment. For example, the 1994 NEHRP Recommended Provisions contains a new *Importance Factor* I_p and revised seismic force and displacement equations for nonstructural components developed based on recent actual earthquake performance data, which were not considered in the preceding NEHRP document (1991). However, most of the new design requirements/recommendations for nonstructural components have only recently been included in the building codes (1994 editions). The effectiveness of these new requirements/recommendations in limiting damage to nonstructural components is not yet known because implementation of these new provisions has just started, and there has not been a major earthquake since 1994 to allow a thorough assessment.

Numerous instances of damage to nonstructural components during past earthquakes have been reported[1,2,3,4]. Typical damage includes failures of suspended ceilings, lighting fixtures, piping systems, mounting fixtures and anchors for equipment, claddding, partitions, etc. Much of this damage reportedly resulted from two main reasons aside from being the direct result of structural failure. One reason is the incompatibility of movement between the building and nonstructural components and also between different nonstructural components which were installed in close proximity to one another. The other reason is the inadequacy of nonstructural components (mainly the support conditions and the mounting fixtures) to sustain seismic lateral load.

Among instances of damage due to displacement incompatibility are damage to suspended ceilings and other components located in or above suspended ceilings in commercial office buildings. These components include the suspended acoustical tile ceiling itself; fire sprinkler systems; light fixtures; and HVAC ducts. Problems arise because these components are co-located in the ceiling area, and their movements during an earthquake are often incompatible due to differences in the component flexibility. For example, fire sprinkler heads usually project through suspended acoustical tile ceilings. During an earthquake, if the movements of the suspended ceiling and the sprinkler pipes are incompatible, either the sprinkler heads or the suspended ceiling, or both, will be damaged. Not only does this decrease the ability of the sprinkler system to suppress post-earthquake fires, but it also may result in broken sprinkler heads and flooding of the building contents. Another example is distortion of the acoustical tile ceiling grid, which may cause lighting fixtures and ventilation grills resting in the grid to fall.

Examples of damage due to the inadequacy of nonstructural components to sustain seismic lateral force include failures of anchors to hold equipment in place, such as water tanks and boilers. This results in equipment sliding off supports, spilling of contents, and disruption of service. Other nonstructural damage of this type includes failure of light fixtures and suspended ceilings at points of connection, and cracks in partition walls.

1.2 Aims of Research

The significant economic impact resulting from nonstructural components damage, evidenced in every past major earthquake, has emphasized the need for mitigation of damage to these building components. Thus, in an effort to address the problem of earthquake damage to nonstructural building components, this research study was initiated. The overall objectives of this study are to examine in detail the nature of failure of nonstructural components, to assess the adequacy of various codes concerning seismic design of nonstructural components, and to propose design guidelines to ensure adequate seismic performance of these components.

1.3 Objectives and Scope of Report

1.3.1 *Objectives*

The objectives of this report are to assess the current state of knowledge in seismic design of nonstructural building components, as reflected in various model building codes currently in use in the United States and other countries, and to identify areas of needed research contributing to the development of methods for reducing seismic damage to nonstructural components of buildings.

1.3.2 *Scope*

This report consists of the following chapters:

Chapter 1 describes in general terms the problem of damage to nonstructural building components and the objectives of this study.

Chapter 2 summarizes documented cases of damage to nonstructural components during past earthquakes.

Chapter 3 summarizes the seismic design requirements for nonstructural building components contained in various model building codes of countries in seismic regions, including the Uniform Building Code[6], the Standard Building code[7], the New Zealand Standard[8], the NEHRP recommended provisions[9,10], and the seismic building codes of Japan[11,12].

Chapter 4 describes the maximum seismic design requirements for various nonstructural components contained in different building codes, and compares these requirements.

Chapter 5 summarizes the findings.

2. OBSERVED NONSTRUCTURAL DAMAGE, RELATED STUDIES AND STANDARD PRACTICES

Damage to all types of nonstructural building components have been observed and reported in every recent major earthquake, including the 1994 Northridge earthquake[4], the 1989 Loma Prieta earthquake[3,29], the 1987 Whittier Narrows earthquake[13], the 1983 Coalinga Earthquake[31], the 1971 San Fernando earthquake[1,2], and the 1964 Alaska earthquake[30]. This nonstructural damage, especially the damage which occurred during the more recent earthquakes such as the 1994 Northridge and the 1989 Loma Prieta earthquakes, highlights the continuing problem posed by deficient seismic performance of nonstructural building components, and emphasizes the urgent need to address these deficiencies.

Surveys of damage to nonstructural building components are frequently conducted by various organizations, including universities, professional engineering organizations, consulting firms, etc. Information on damage obtained from these post earthquake surveys is usually made available in professional journals, conference papers, and internal reports. Since the focus of this report is on the seismic performance of nonstructural ceiling components, only the information relevant to seismic performance of these components is extracted from these publications to reveal typical damage sustained by ceiling components due to earthquakes. Since this information is extracted from other surveys, the distinction between *damage* and *failure* can not be made with confidence. Therefore, both damage and failure are referred to as *damage* in this chapter.

2.1 Observed Ceiling Components Damage

In this section, a review is presented of typical damage to nonstructural ceiling components observed in the most recent major earthquakes in the U.S. These observations are presented to illustrate the need for damage mitigation techniques for nonstructural components located within or above the ceiling area: *suspended ceilings, lighting fixtures, plaster or sheetrock ceilings, and fire protection sprinkler systems.*

2.1.1 *Suspended Ceilings*

Probably the most widely reported type of nonstructural damage in earthquakes is the failure of suspended ceiling systems. Even under moderate shaking, the lightweight acoustical tile panels which rest on the main and cross runners of suspended ceilings are easily dislodged. Although this is often frightening and disorienting to building occupants, it seldom causes injury or serious damage. Furthermore, the panels can often be easily re-installed, or replaced at a relatively low cost. Still, it hardly seems necessary that the loss of suspended ceiling panels should be a regular occurrence in earthquakes. Inexpensive and effective methods for retaining tiles and stabilizing the ceiling grid are available.

A more serious type of damage associated with suspended ceilings is the collapse of light fixtures and air diffusers incorporated into the ceiling system. Very often light fixtures are the fluorescent type, containing heavy electrical ballasts (transformers) and metal or plastic light diffusers. Even if a light fixture stays in place, the diffuser and even the light bulbs may drop, potentially causing injuries. Ceiling light fixtures not only present a falling debris hazard, but the loss of interior lighting can also hinder evacuation and rescue efforts, and can render a building unusable until the fixtures are replaced. The seismic performance of lighting fixtures is discussed further in section 2.1.2 "Lighting Fixtures."

Damage to suspended ceilings has been reported following almost every moderate to large U.S. earthquake in modern times. Fallen suspended ceilings often attract a great deal of attention because their appearance is spectacular. The damage actually caused by fallen ceiling tiles is often minimal, although the tiles can damage delicate equipment and block ingress and egress routes. Perhaps the most significant aspect of ceiling performance is the damage caused when the ceiling system as a whole interacts with other structural and nonstructural elements such as pipes, ducts, partitions and beams.

Sharpe et al. (1973)[40] noted that a suspended ceiling in a control room at an electrical power plant completely collapsed (both the panels and the support grid fell) during the 1971 San Fernando earthquake. The fallen ceiling did not cause any injuries or major equipment damage, but it did obstruct the use of the control room in the critical hours following the earthquake.

In the 1987 Whittier Narrows earthquake most of the ceiling tiles fell, and 75 percent of the suspended ceiling system was destroyed in Salazar Hall at California State University, Los Angeles (Taly 1988). Falling ceiling tiles also damaged personal computers. Acoustical baffles fell from the ceiling of the Physical Education and Gymnasium building. There were no injuries, but the gymnasium floor was damaged.

In the 1989 Loma Prieta earthquake widespread damage to suspended ceilings was observed (EERI 1990). This was attributed to the lack of lateral bracing of ceiling grids, and inadequate connections at the ceiling/wall interface. Damage was concentrated at the ceiling perimeters and corners. In a survey of computer facilities in the San Francisco Bay area, it was found that ceiling tiles fell in about 40% of the facilities, but that no resulting damage or injuries were reported.

Griffin and Tong (1992)[34] summarized the seismic performance of suspended ceilings at electrical power plants and industrial facilities in the 1987 Whittier Narrows and 1989 Loma Prieta earthquakes. It was found that even ceilings designed according to the current UBC standards were damaged in these events. The main cause of damage was inadequate lateral restraint of the ceiling plane, which permitted the ceiling to swing like a pendulum. Ceilings braced with diagonal wires, as specified in the UBC standards, were inadequately restrained because there was always some slack in the restraining wires. Because the ceilings were permitted to move horizontally, they impacted walls and columns, causing the ceiling panels to drop and grid frame members to buckle. The ceilings which performed the best had their support grids anchored to the perimeter walls, thus limiting lateral movement. Damage was also caused by differential movements between suspended ceilings and piping and HVAC systems which penetrated the ceiling. Griffin and Tong (1992)[34] present a series of recommended practices, some more stringent than current standards, which are intended to improve the seismic performance of suspended ceiling systems.

Observations by EERI (1994)[43] following the 1994 Northridge earthquake indicated that suspended ceilings which had been designed according to current code requirements generally fared well, except in the regions of strongest ground shaking. Damage was also observed in ceilings further from the epicenter which were designed according to current codes, but which had been constructed without some diagonal bracing wires because of interferences between the wires and ducts or piping.

Laboratory tests of sections of ceilings in combination with nonstructural partition walls were conducted by Rihal and Granneman (1984)[28]. It was determined that a major factor influencing seismic damage to suspended ceilings is the connection details for the ceiling perimeter. When ceilings were unattached to the perimeter walls the observed damage was much greater than when the ceilings were anchored to the perimeter walls. These tests were conducted on sections of ceilings measuring only 3.6 m by 4.9 m (12 ft by 16 ft), so it is not clear how the conclusions would apply to much larger ceiling areas.

2.1.2 *Light Fixtures*

Since commercial light fixtures typically weigh 5 to 25 kg (11 to 55 lbs), the collapse of ceiling mounted fixtures in an earthquake can pose a significant safety hazard. Even if light fixtures do not cause injuries when they fall, the resulting tangle of broken light fixtures can cause confusion and panic among building occupants. Furthermore, in the period of recovery and cleanup after an earthquake, functional interior lighting is a prerequisite to returning the building to its normal use.

Light fixtures have often fared poorly in past earthquakes, particularly pendant mounted fixtures and fixtures mounted in lightweight suspended ceilings. In a single office building it is not uncommon for dozens, or even hundreds of fixtures to drop from their mountings or lose their diffuser grills during an earthquake.

There are far too many instances of lighting fixture damage in past earthquakes to give any comprehensive listing here. However, the following examples are typical.

In the 1987 Whittier Narrows earthquake pendant lighting fixtures fell to the floor in the Gymnasium Building of California State University, Los Angeles (Taly 1988)[13]. The falling fixtures caused no injuries, but the gymnasium floor was damaged.

In the 1989 Newcastle, Australia earthquake it was reported that hundreds of light fixtures, weighing up to 22 kg (48 lbs) each, fell in industrial plants in the area (Melchers 1990)[37]. Since the earthquake occurred over a holiday, no one was injured in the plants. At the time of the earthquake no seismic building code was in effect in Newcastle, so the light fixtures had not been secured against earthquakes.

In the 1994 Northridge earthquake it was observed that fluorescent light fixtures tethered by two "fail-safe" support wires in suspended ceilings at the Olive View Medical Center were prevented from falling more than a few centimeters (EERI 1994)[43]. However, in other locations, such as the Levitz and Carpeteria stores in Northridge, fluorescent fixtures without tether wires fell either to the ground, or to the limits of the electrical conduit, remaining suspended by the conduit a meter or more below the ceiling level.

Pendant mounted ceiling light fixtures fell to the floor or on top of desks in about 100 public school classrooms during the 1994 Northridge earthquake (EERI 1994)[43]. These fixtures, weighing as much as 35 kg (75 pounds), were in older facilities, so they had been installed without the safety tether wires required by current codes.

2.1.3 *Plaster Ceilings*

Heavy plaster ceilings, some including decorative details, are common in many older buildings. These ceilings are usually not anchored well to the framing system above, and they are susceptible to seismic damage. Often the plaster is held in place only by adhesion and limited mechanical interlock with underlying lath or wire mesh. The lath or mesh may in turn be suspended by a framework of light gage steel (cold formed or rolled sections), which is itself subject to failure. Large pieces of plaster, weighing tens of kilograms, or even entire sections of ceiling weighing much more, can fall onto building occupants. In some cases, the fasteners which hold lath or wire mesh to the framing system, or a light gage framework supporting a ceiling, can become corroded over time, further weakening the system and making it more susceptible to earthquake damage.

Another location where heavy plaster ceilings are found is on the exterior soffits of buildings. Failure of soffits at building exits can be particularly hazardous since the ceiling debris will fall on occupants attempting to escape from the building during an earthquake.

A classic example of a fallen plaster ceiling was the collapse of the ceiling in the Geary Theater in San Francisco during the 1989 Loma Prieta Earthquake (Lagorio 1990[36], EERI 1990[29]). The first six rows of the 1350 seat theater were covered with slabs of plaster and theater lighting fixtures. There were no injuries because at the time of the earthquake the theater was not occupied.

The U.S. Court of Appeals Building in San Francisco was built in 1905. It was damaged by the 1906 San Francisco earthquake, but survived. In the 1989 Loma Prieta earthquake the building suffered additional damage. The interior of the building is highly decorated, including ornate molded plaster ceilings. It was partly out of a desire to conserve the fine architectural detailing of the plaster ceilings that an extensive seismic retrofit program was undertaken for the building, including shear wall strengthening and base isolation.

In the 1989 Loma Prieta earthquake, tile mosaics were damaged on the domed ceiling of the Memorial Church at Stanford University (Poland and Reis 1992)[38]. The weak mortar behind the mosaics was replaced with a fiberglass resin anchorage system. Rather than entirely remove the mosaics, replace the mortar, then re-install the mosaics, fiberglass molds were cast against the faces of the mosaics and the backing mortar was removed by working from the back side of the mosaics.

In the 1994 Northridge earthquake it was reported (EERI 1994)[43] that a 4.5 m by 23 m (15 ft by 75 ft) portion of a heavy plaster ceiling collapsed in an auditorium at the University of California, Los Angeles (UCLA). The heavy debris fell directly onto the seating area, which was unoccupied at the time of the earthquake. The cause of the collapse was reported as the failure of welds in thin gage steel channel hangers.

EERI (1994)[43] also reported on the failure of two exterior suspended plaster building soffits over entrances. In one case the soffit fell over the main entrance of the Oviatt Library at California State University, Northridge. In the other case the soffits from two levels of the Cygna Health Plan Building fell over one of the building entrances.

2.1.4 *Fire Protection Sprinkler Systems*

The most commonly employed fire suppression system for large building areas is the automatic sprinkler system, triggered by heat or smoke sensors. Since fires are often caused by earthquakes, it is essential that sprinkler systems be in perfect working order following an earthquake. Despite the importance of fire suppression systems, their seismic performance is often marginal.

The most common problem with fire suppression systems has been the interaction of the piping and sprinkler system with other building elements, particularly suspended ceilings. Sprinkler nozzles are usually mounted on the ends of pendant pipes, which project through a suspended ceiling. Differential movement between the suspended ceiling and the sprinkler pipe system often damages the pendant pipes and the sprinkler heads. If pipe damage causes a water leak, extensive water damage to building contents and finishes may result, particularly if the water cannot be shut off because the building is unsafe to occupy immediately following an earthquake. If sprinkler nozzles are damaged the spray coverage is reduced, which compromises the effectiveness of the fire suppression system. Damage to the piping system can also occur in long, straight runs of unbraced pipe. Since a long run of pipe is flexible, it tends to oscillate widely under earthquake shaking, and can collide with other building elements such as other pipes, ducts, wiring, and the structural frame.

Harris (1992)[35] describes the development of the seismic provisions of the National Fire Protection Association Standard 13 (NFPA 1994b)[19], which governs the installation of fire sprinkler piping systems. Seismic considerations have been included in NFPA-13 since the 1940s, and have evolved to their present form, which includes provisions for lateral bracing of pipes and flexible connections at critical locations.

A proposed seismic design method for rod hanger systems was presented by Conoscente and Eder (1990)[33]. Rod hangers are used to suspend piping networks, cable raceways, ducts and electrical conduits. The proposed design method is based on the rod anchor capacity and the fatigue life of the rod, rather than on a simplified equivalent static lateral load procedure, as advocated by current codes.

One approach to protecting piping systems from earthquakes is to provide rigid lateral bracing at intervals along the pipe. However, rigid restraint of piping systems is often at odds with the requirement that the piping system be able to expand and contract with changes in temperature. One solution to this dilemma was outlined by Shimoda et al. (1992)[41]. A support device was proposed which consists of a semiactive damper driven by a piezoelectric actuator. The damper would not restrict gradual thermal movements of the pipe, but would become active under earthquake accelerations. An analytical study of the proposed damper indicated that it could control pipe motions with a piezoelectric actuator force of less than 50 N.

Scawthorn et al. (1992)[39] described the failure of the fire sprinkler system in the California Federal Data Processing Center during the 1987 Whittier Narrows earthquake. This was a four-story steel frame building, with computer facilities on the ground floor and office facilities on the top three floors. During the earthquake, smoke detectors were triggered either by dust in the air, or by the shock the detectors received when they fell along with parts of the suspended ceiling system. Thus, even though there were no fires, sprinkler systems were activated throughout the building. Before the sprinklers could be shut off, enough water was released to cause extensive damage. Water in the upper floors collected in electrical cable troughs, and was routed to the lower floors through electrical service penetrations. Computer equipment on the ground floor was protected by plastic sheets which employees spread over the cabinets. The cost of water damage in this building far outweighed the cost of damage caused directly by ground shaking.

Harris (1992)[35] reported that in the Loma Prieta earthquake, in areas with Modified Mercally Intensity of VII to VIII, the failure rates of sprinkler systems was between 5% and 10%. Harris reviewed the damage to sprinkler systems which has been observed in the U.S. in the period between the 1964 Alaska earthquake and the 1989 Loma Prieta earthquake. Commonly observed shortcomings have included the following: damage to sprinkler heads, due to impact with other building elements or interaction with gypsum board ceilings or suspended ceilings; failures of branch line pipes which are suspended in U-shaped hangers (which provide no uplift restraint) or by "C" clamps and rods (the "C" clamps tend to lose their clamping force under dynamic loading); pipe ruptures due to inadequate flexibility in the pipe system or inadequate clearances to accommodate structure movements; failures of pipe anchorage systems, particularly those mounted with powder-actuated fasteners; and general system failure due to rupture of underground supply mains in areas with soft soil conditions.

Arnold (1991)[32] reported on a 1990 survey by the Building Owners and Managers Association of San Francisco on damage suffered by 129 medium and large office buildings in the 1989 Loma Prieta earthquake. It was found that the median cost of damage per building was approximately $95,000, and that about $50,000, or 53%, of this cost was due to water damage from fire sprinkler systems. Arnold also reported that in the Loma Prieta earthquake a sprinkler system failure at the North Terminal of the San Francisco International Airport resulted in "several million dollars of property damage and a shut-down of the airport for several hours due to malfunctioning sprinklers."

In the 1989 Loma Prieta earthquake damage was sustained by a fire sprinkler system at a computer equipment manufacturing plant in Watsonville, as reported by EERI (1990)[29]. Sprinkler heads were damaged in two buildings, and the heads sprayed water onto the surrounding area. In one building the sprinkler head damage was due to impacting of the sprinkler system and the suspended ceiling system; in the other building the damage was due to the sprinkler heads impacting the timber ceiling beams. In one location a damaged sprinkler head sprayed water onto an electrical transformer and switch gear equipment. Since power was lost during the earthquake, there was no damage to the electrical equipment, although it was out of service for several days while it was disassembled and dried. In addition to the sprinkler head damage, the joints of fire sprinkler pipes cracked and leaked at two locations, due to insufficient lateral restraint of the pipe.

A striking example of the effect of fire suppression systems on building occupancy is the performance of the Olive View Medical Center in the 1994 Northridge earthquake. In 1971 Olive View suffered severe structural damage during the San Fernando Earthquake. The hospital was demolished and rebuilt under enhanced structural design requirements (the California Hospital Act of 1972). In the 1994 earthquake the building performed extremely well from a structural standpoint, even though the ground accelerations recorded near and on the structure were unusually high. The hospital had to be abandoned, however, following the earthquake, in part because the fire sprinkler system and chilled water lines had been damaged, resulting in water leaks. The building had to be evacuated for several days while repairs and cleanup were undertaken (Todd et al., 1994[42], EERI 1994[43]). Thus, although the structure itself performed very well, the non-structural fire suppression system and chilled water system performed poorly, rendering the structure unusable. This was a particularly serious result in this case, as hospitals are in critical demand following earthquakes.

2.2 Related Experimental Studies

Two related studies were identified in the literature and are summarized in this chapter. Both are experimental programs dealing with suspended ceiling systems. One study was conducted by Anco Engineers Inc.[24] and one by Rihal and Granneman[28]. Other literature relevant the seismic performance of nonstructural building components includes studies by Clark and Glogau[25], Meehan[26], and KRTA Limited[27]. These studies are cited in the reference for information but not reviewed here since they provide only general discussions of codes and proposed methods for seismic restraints of ceiling systems rather than specific experimental data.

2.2.1 *Tests by Anco Engineers Inc (1983)*[24]

Noting that "*ceilings and lighting fixtures are always one of the nonstructural elements noted during earthquake damage surveys*", Anco Engineers Inc. conducted a shake table test program to "*evaluate the seismic restraint of an entire ceiling finish system with adjunct light fixtures*". The type of ceiling system studied was *Direct-hung, T-bar grid suspended acoustic lay-in tile ceiling system*, as shown in Figure 2.1.

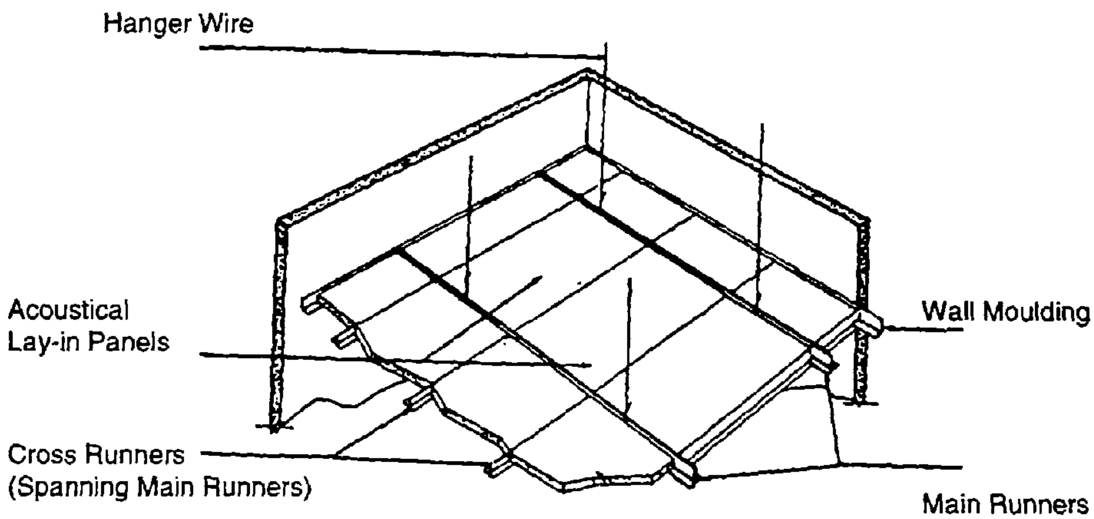

Figure 2.1 Direct-hung suspended ceiling system

A prototypical 3.66m x 8.53m (12 ft x 28 ft) ceiling area which was comprised of 1.22m x 1.22m (4 ft x 4 ft) standard suspension modules was used for testing. The suspension modules consisted of intermediate duty (ASTM C635), suspended main and cross tee rails with lay-in acoustical tiles and lighting fixtures. The ceiling system was tested using an overhead shake table with the Taft strong ground motion selected as input motion. Various constraint configurations were considered, including:

1. Without wall perimeter:
   - Free-free pendulum without restraints
   - 45° splay wire seismic restraints (in accordance with 1982 UBC)
   - 45° splay wire with center post or strut.

2. With wall perimeter:
   - 45° splay wire with center post or strut
   - 45° splay wire restraint
   - Pop rivet attachment, at 0.61m (2 ft) and 1.22m (4 ft) spacings, along perimeter wall.

Three specific test results reported by Anco Engineers Inc. are:

1. The 1982 UBC provision requiring the installation of a vertical compression strut at splay wire restraint points do not appear to be justified since no appreciable difference in ceiling dynamic behavior for the cases of with and without vertical struts was observed.

2. Pop rivet attachments, often placed for alignment purposes, prevent the 45° splay wires from acting as seismic restraints and become the defacto seismic restraints as the splayed wire bracing is too flexible.

3. Safety wires on drop-in light fixtures are a simple and cost-effective seismic hazard mitigator. Two slack safety wires are recommended to be attached to diagonal corners of light fixtures and directly attached to the structure above (structural ceiling) to prevent drop-out.

2.2.2 *Tests by Rihal & Granneman (1984)*[28]

The objectives of this dynamic test program were to investigate the following:

1. The behavior of unbraced and braced (by splayed wires) suspended ceilings without partitions.

2. The behavior of partitions subjected to motions normal to the plane of the partitions.

3. The behavior of suspended ceilings (unbraced and braced by splayed wires), and interaction between partial-height partitions and suspended ceilings.

4. The behavior of suspended ceilings (unbraced and braced) and interaction with full-height partitions, including effects of perimeter detailing.

5. The behavior of braced suspended ceilings with and without a vertical pipe strut at the point of bracing.

The suspended ceiling specimens are typical tee-grid ceilings with lay-in acoustical tiles, 3.66m by 4.88m (12 ft x 16 ft) in plan. The grid consisted of main and cross tees at 1.22m (4 ft) and 0.61m (2 ft) centers, respectively. The partial-height partitions are 2.44m (8 ft) high, and the full-height partitions are 3.05m (10 ft) high. The partial-height partitions are fixed to the ceiling grid while full-height partitions pass through the grid to the structure above. Building partitions are framed with gypsum wallboard as facing material. The partition assembly consists of horizontal metal runners at base and at top.

Input motions for dynamic testing were sinusoidal and introduced by means of a shake table. Three types of dynamic tests were used: (1) *Damping Test* (the specimen was excited through one-half cycle of sinusoidal motion of 5 Hz frequency which was then allowed to decay); (2) *Sine Sweep Test* (peak displacement was fixed and frequency was varied); and (3) *Block Cyclic Test* (the input motion frequency was fixed and peak displacement varied).

The following results were reported by this test program:

1. Behavior of building partitions and suspended ceilings is influenced both by acceleration and displacement levels and the frequency of the input motion.

2. 1982 UBC provision of the 45° splayed wire ceiling bracing is satisfactory for providing stability to partial-height partitions and should be enforced for all building.

3. Specimens with splayed wire bracing and vertical strut seemed to be initially stiffer than the specimens without vertical strut.

4. Specimens with vertical strut at point of splayed wire bracing have less ceiling uplift than those without vertical strut.

5. Ceiling perimeter details (partition and soffit) are a predominant factor influencing damage to suspended ceilings and partitions.

6. Extensive damage to suspended ceilings occurred at the unattached ceiling perimeter (soffit end) at frequencies between 4 to 4.8 Hz, and peak displacement of 25mm (1 inch).

7. Addition of vertical suspension wires prevented ceiling tiles from falling.

8. Partition damage was limited to loosening of drywall screws.

2.3 Standard Practices

Standard Practices are usually published by building trade organizations in collaboration with building hardware manufacturers and professional standards organizations. These documents typically provide guidance for the selection of hardware and for appropriate installation of the selected materials. The requirements outlined in these Standard Practices are supposed to reflect the design requirements prescribed by the building codes in effect. Unlike the building codes which prescribe general engineering requirements and are used mostly by the engineering and design professions, Standard Practices provide specific instructions for specific building trades. A number of Standard Practices related to ceiling components have been identified in this study. They are listed here for reference:

- National Fire Protection Association, *Installation of Sprinkler Systems*, NEPA 13, 1991.

- American Society for Testing and Materials, *Standards Practice for Installation of Metal Ceiling Suspension Systems for Acoustical Tile and Lay-in Panels*, ASTM C636-92.

- American Society for Testing and Materials, *Standard Practice for Application of Ceiling Suspension Systems for Acoustical Tile and Lay-in Panels in Areas Requiring Seismic Restraint*, ASTM E580-91.

- American Society for Testing and Materials, *Standard Specification for the Manufacture, Performance, and Testing of Metal Suspension Systems for Acoustical Tile and Lay-in Panel Ceilings*, ASTM C635-95.

- Manufacturers Standardization Society of the Valve and Fittings Industry, Inc. (MSS), *Standard Practice for Pipe Hangers and Supports- Selection and Application*, MSS- SP-69.

- City of Los Angeles, Department of Building and Safety - Rules of General Application, RGA 4-74, *Recommended Standards for Suspended Ceilings Assembles*.

- City of Los Angeles, Department of Building and Safety, Rules of General Application, RGA 12-69, *Standards for Lighting Fixture Supports*.

- Ceilings and Interior Systems Construction Association (CISCA), *Guidelines for Seismic Restraint, Direct Hung Suspended Ceilings Assemblies, Seismic Zones 3 and 4.*

3. CODES FOR SEISMIC DESIGN OF NONSTRUCTURAL COMPONENTS

This chapter summarizes the seismic requirements for the design of nonstructural components, in terms of lateral force and displacement, of four building codes with earthquake provisions currently in use in three countries; the United States, New Zealand, and Japan. In the U.S., the codes reviewed include two of the national model codes, namely the 1994 Uniform Building Code[6] (1994 UBC) of the International Conference of Building Officials, and the 1994 Standard Building Code[7] (1994 SBC) of the Southern Building Code Congress International. In New Zealand, the 1992 New Zealand Standard[8] (NZS 4203: 1992) is reviewed. And in Japan, two seismic building codes[11,12] are reviewed: (1) the Guideline for Seismic Design of Building Nonstructural Components published by Public Buildings Association in 1987, and (2) Guideline for Seismic Design of Building Equipment published by Building Center of Japan in 1984. Also reviewed is the 1994 NEHRP Recommended Provisions for Seismic Regulations for New Buildings[9,10] (1994 NEHRP). The NEHRP Recommended Provisions contain seismic design provisions to be considered for adoption in a future version of the national model codes. The review of codes and recommended provisions is intended to reveal the variation in seismic design requirements for nonstructural components between different codes of practice. Provisions and design information relevant to the design of nonstructural components of each of the above listed documents are summarized below.
June 13, 1996

3.1 Uniform Building Code[6] (UBC 1994)

3.1.1 *Seismic Force Requirement*

Permanent nonstructural components and their attachments shall be designed to resist the total design lateral seismic force, F_p, prescribed below:

$$F_p = ZI_pC_pW_p \qquad \text{(UBC 1994 equation 30-1)}$$

where:

Z is the **seismic zone factor**, which ranges between 0.075 (seismic zone 1) to 0.40 (seismic zone 4). Values of Z for different seismic zones and seismic zone designation are given in Appendix A, Table A.1 (UBC 1994 Table 16-I) and Figure A.1 (UBC 1994 Figure 16-2).

I_p is the **seismic importance factor** for nonstructural components and their attachments. Values of I_p corresponding to different UBC **occupancy categories** are listed in Appendix A, Table A.2 (UBC 1994 Table 16-K).

C_p is the **horizontal force factor**. Values of C_p for nonstructural components are given in Appendix A, Table A.3 (UBC 1994 Table 16-O). C_p varies between 0.75 (for most mechanical and electrical equipment) to 2.0 (for exterior and interior ornamentation and appendages, signs and billboards, etc.).

W_p the **weight** of an element or component.

3.1.2 *Seismic Displacement Requirement*

UBC 1994 is less specific with requirements for seismic displacements of nonstructural components. In general, UBC 1994 requires that, for *Essential* and *Hazardous Facilities* (building categories 1 and 2 in Table A.2), the lateral-force design shall consider the effects of relative motion of the points of attachment to the structure.

3.2 Standard Building Code[7] (SBC 1994)

3.2.1 *Seismic Force Requirement*

The 1994 SBC provides two different sets of seismic force requirement for nonstructural components, one for architectural components and their attachments, and one for mechanical, electrical components and their attachments.

For architectural components and their attachments, the design seismic force F_p is determined as follows:

$$F_p = A_v C_c P W_c$$

For mechanical, electrical components and their attachments, the design seismic force F_p is determined as follows:

$$F_p = A_v C_c P a_c W_c$$

where:

A_v = **The effective peak velocity-related acceleration** ($0.05 \leq A_v \leq 0.40$), to be selected from 1994 SBC Figure 1607.1.5A, *Contour Map of Effective Peak Velocity-Related Acceleration Coefficient.* This map is reproduced in Appendix B, Figure B.1.

C_c = **Seismic Coefficient** from 1994 SBC Tables 1607.6.3 (for architectural components and their attachments) and 1607.6.4A (for mechanical, electrical components and their attachments). Tables 1607.6.3 and 1607.6.4A are reproduced in Appendix B, Tables B.1 and B.2.

P = **Performance criteria factor**, varied with **seismic hazard exposure group** (I to III) and determined from 1994 SBC Tables 1607.6.3 for architectural components and their attachments (Appendix B, Table B.1) and 1607.6.4A for mechanical, electrical components and their attachments (Appendix B, Table B.2). Table B.3 of Appendix B (1994 SBC Table 1607.1.6) list the seismic hazard exposure group for buildings.

a_c = **Attachment amplification factor**, determined in accordance with 1994 SBC Table 1607.6.4B (Table B.4, Appendix B). a_c is either 1.0 or 2.0.

W_c = **The operating weight** of the nonstructural component.

3.2.2 *Seismic Displacement Requirement*

The 1994 SBC provides two sections dealing with seismic displacement requirements for nonstructural building components, one for *architectural* components (1994 SBC section 1607.6.3.2) and the other for the attachment of *mechanical and electrical* (1994 SBC section 1607.6.4.2) components. For *architectural components*, deformation due to design story drift Δ_m, computed as the difference between story-level displacements δ_{xm}, shall be considered. δ_{xm} is computed as follows (1994 SBC section 1607.5.6):

$$\delta_{xm} = C_d \delta_{xem}$$

and,

$$\delta_{xem} = (g/4\pi^2)(T_m^{\,2} F_{xm}/w_x)$$

Where:

δ_{xem} = The deflection of level x in the m[th] mode at the center of the mass at level x.
C_d = The deflection amplification factor (given in 1994 SBC Table 1607.3.3).
g = The acceleration due to gravity (feet per second^2).
T_m = The modal period of vibration, in seconds, of the m[th] mode of the building.
F_{xm} = The portion of the seismic base shear in the m[th] mode, induced at level x.
w_x= The portion of the total gravity load of the building, W, located or assigned to level x.

For *mechanical and electrical* components, the 1994 SBC requires that relative seismic displacements between two points of support (or points of attachment) of these components be considered and accommodated. In calculating the relative seismic displacements between points of support, the difference in elevation between the supports and the out-of-phase displacements across portions of the building that are capable of moving in a differential manner such as at seismic and expansion joints, are to be considered (1994 SBC section 1607.6.4.2). Displacements at points of support shall be computed as described above.

$$\delta_{xem} = (g/4\pi^2)(T^2_m F_{xm}/W_x)$$

Where:

δ_{xem} = The deflection of level x in the m[th] mode at the center of the mass at level x.
g = The acceleration due to gravity (feet per second^2).
T_m = The modal period of vibration, in seconds, of the m[th] mode of the building.
F_{xm} = The portion of the seismic base shear in the m[th] mode, induced at level x.
W_x = The portion of the total gravity load of the building, W, located or assigned to level x.

3.3 New Zealand Standard[8] (NZS 4203: 1992)

3.3.1 *Seismic Force Requirement*

The horizontal seismic force, F_{ph}, on nonstructural components of a building (referred to as **parts** of the building) shall be determined from:

$$F_{ph} = C_{ph} W_p R_p \qquad \text{(NZS equation 4.12.1)}$$

The vertical seismic force, F_{pv}, on nonstructural components of a building shall be determined from:

$$F_{pv} = C_{pv} W_p R_p \qquad \text{(NZS equation 4.12.2)}$$

where:

W_p = **Weight** of the nonstructural components or their attachments.

R_p = **Risk factor** for nonstructural components or attachments, listed in Appendix C, Table C.1 (NZS Tables 2.3.2 and 4.12.1).

C_{ph} = **Seismic coefficient**, shall be taken equal to the **basic horizontal coefficient** C_{pi} (basic horizontal coefficient for nonstructural components and attachments at level i). C_{pi} shall be computed as follows:

$$C_{pi} = C_b(T_{pe}, \mu_p) C_{fi}/0.4 \qquad \text{(NZS equation 4.12.7)}$$

and,

μ_p = **Structural ductility factor** for the nonstructural components. Values$_p$ for μ corresponding to various nonstructural component are listed in Table C.2 (NZS 4203 Table C4.12.1). μ_p = 1.0 for connections for machinery, switch gear and the like.

$C_b(T_{pe}, \mu_p)$ = **Basic seismic acceleration coefficient** for intermediate soil and T_{pe} is the equivalent period of the nonstructural components given by = $0.2\ T_p/T_1$ but not to be taken less than 0.4 s. T_1 is the **fundamental translational period of vibration** of the structure (NZS 4203 Section 4.5.2). Tabulated values of C_b corresponding to different values of T_p and μ_p are given in Appendix C, Table C.3 (NZS 4203 Table 4.6.1).

C_{fi} = **Floor acceleration coefficient** at levels between the building base and the uppermost principal seismic weight. C_{fi} shall be determined by either the **equivalent static method** (NZS equation 4.12.5) or the **modal response spectrum method** (NZS equation 4.12.6) listed below:

$$C_{fi} = \frac{C_b(T_1, \mu_o)}{C_b(T_1, 1)} C_{fo}(1 - h_i/h_n) + C_{fn}(h_i/h_n) \qquad \text{(NZS 4203 equation 4.12.5)}$$

or

$$C_{fi} = \frac{C_b(T_1, \mu_o)}{C_b(T_1, \mu)} \frac{F_i}{W_i} \qquad \text{(NZS 4203 equation 4.12.6)}$$

μ_0 = **Structural ductility factor** calculated using the overstrength values (NZS 4203 Section 4.12.2). C_{fo} is the **floor acceleration coefficient** at and below the base of the building. C_{fo} may be computed as follows:

$C_{fo} = 0.4\, RZL_s$ for the serviceability limit state (NZS equation 4.12.3(a))
$= 0.4\, RZL_u$ for the ultimate limit state (NZS 4.12.3(b))

C_{fn} = **Floor acceleration coefficient** at the level of the uppermost principal seismic weight. C_{fn} may be computed as follows:

$$C_{fn} = \frac{C_b(T_1,\mu_o)}{C_b(T_1,\mu)} \frac{F_n}{W_n}$$

R : **Building risk factor**, listed in Table C.4 (NZS Tables 2.3.1 and 4.6.3).
Z : **Zone factor** ($0.4 \le Z \le 0.8$), shown in Appendix C, Figure C.1 (NZS Figure 4.6.2).
L_s,L_u : **Limit state factors** for serviceability state (1/6) and ultimate state (1.0), respectively.
F_i : **Equivalent static lateral force** at level i; or inertial force at level i found from combination of modal inertial force.
F_n : **Inertial force** at the height of the uppermost principal seismic weight, h_n.
h_i : **Height** of level i above the level where the ground provides lateral restraint to the structure.
h_n : **Height** from the base of the building to the level of the uppermost principal seismic weight.
C_{pv} : taken as RZL_s for the serviceability limit state and RZL_u for the ultimate limit state.

An alternate method for obtaining C_{ph} (or C_{pi}) without having to use NZS equation 4.12.7 is to read the normalized values of C_{ph} from Table C.5 in Appendix C (NZS Table C4.12.2). These values were calculated for the following assumptions:

- Each structure has equal story heights and weights.
- The fundamental period, T_1, is not less than the greater of 0.6 s and 0.10n, where n is the number of stories.
- The nonstructural components, with their connections, are stiff ($T_p = 0$, $T_{pe} = 0.45$ s).
- The structures are sited on flexible or deep soil sites.

3.3.2 *Seismic Displacement Requirement*

NZS 4203's seismic displacement requirement is less specific than the seismic force requirement. In general, NZS requires that "*deflections of parts (nonstructural components) under the prescribed seismic forces shall be limited so as not to impair their strength or function, or lead to damage to other building components*" (NZS 4203 Section 4.12.1.7). Connections between nonstructural components and the building structure shall be designed to accommodate the interstory deflections determined by either the *equivalent static method* (NZS 4203 section 4.8), the *modal response spectrum method* (NZS 4203 section 4.9), or the *numerical integration time history method* (NZS 4203 section 4.10).

3.4 1994 NEHRP Recommended Provisions for Seismic Regulations for New Buildings[9,10]

The seismic force and displacement requirements for nonstructural building components (including architectural, mechanical, and electrical components) prescribed in this document (1994 edition) were completely revised from the previous edition (NEHRP 1991). The changes include introduction of the **component importance factor I_p** and new force and displacement equations. Specific requirements for relative seismic displacement are recommended and a more rational basis for connection design is provided.

3.4.1 *Seismic Force Requirement*

All nonstructural components and their attachments shall be designed for the seismic force, F_p, centered at the component's center of gravity and distributed relative to the component's mass distribution, described herein (1994 NEHRP section 3.1.3):

$$F_p = 4.0 C_a I_p W_p \qquad \text{(NEHRP eq. 3.1.3-1)}$$

Alternately, F_p may be computed in accordance with the following equations:

$$F_p = a_p A_p I_p W_p / R_p \qquad \text{(NEHRP eq. 3.1.3-2)}$$

and

$$F_{p(minimum)} = 0.5 C_a I_p W_p \qquad \text{(NEHRP eq. 3.1.3-5)}$$

where:

C_a = **Seismic coefficient** at grade, expressed as a fraction of acceleration of gravity and determined based on **Soil profile type** (A to E) and the **effective peak ground acceleration coefficient** A_a (determined from NEHRP seismic ground acceleration map 1). C_a may be obtained as described in NEHRP section 1.4.2.3 or from Table D.1 in appendix D (NEHRP Table 1.4.2.4a). The maximum value of C_a is 0.44 (for soil type D).

I_p = **Component importance factor** which represents the greater of the life-safety importance of the component and the hazard exposure importance of the structure. I_p is either 1.0 or 1.5. I_p is 1.0 for all nonstructural components and their attachments except for those components listed below, in which case I_p is 1.5:

- Life-safety component that is required to function after an earthquake.
- Component contains material that would be significantly hazardous if released.
- Component poses a significant life-safety hazard if separated from primary structure (e.g., parapets, exterior wall panels).
- Component can block a means of egress or exitway if damaged (e.g., exit stairs).

W_p = **Component operating weight.**

a_p = **Component amplification factor** which represents the dynamic amplification of the component relative to the fundamental period of the structure (T). a_p varies from a minimum value of 1.0 to 2.5. NEHRP provides two sets of a_p values, one for architectural components and their attachments as listed in Table D.3 in appendix D (NEHRP Table 3.2.2) and one for mechanical and electrical components and their attachments as listed in Table D.4 (NEHRP Table 3.3.2). In both cases, the value of $a_p = 1$ is for components generally regarded as rigid or rigidly attached. The value of $a_p = 2.5$ is for components generally regarded as flexible or flexibly attached.

R_p = **Component response modification factor** represents the energy absorption capability of the component's structure and attachments. Current R_p values, listed in Table D.3 (NEHRP Tables 3.2.2) for architectural components and their attachments and Table D.4 (NEHRP Table 3.3.2) for mechanical and electrical components and their attachments, are judgmentally determined based on the experience of the responsible committee and vary from 1.5 to 6.0. In general, a higher value of R_p is associated with more ductile materials and detailing.

A_p = **Component acceleration coefficient** at point of attachment to the structure, expressed as a fraction of gravity. A_p may be computed using the following equations (NEHRP equation 3.1.3-3):

$$A_p = C_a + (A_r - C_a)(x/h) \qquad \text{(NEHRP eq. 3.1.3-3)}$$

where:

A_r= **Component acceleration coefficient**, expressed as a fraction of gravity, at the structure roof level. A_r may be computed as follows:

$$A_r = 2.0A_s \leq 4.0C_a \qquad \text{(NEHRP eq. 3.1.3-4)}$$

A_s = **Structure response acceleration coefficient**, expressed as a fraction of gravity. A_s shall be computed for each principle horizontal direction of the structure using the equation listed below. The larger value of A_s shall be used in computing A_r.

$$A_s = 1.2\frac{C_v}{T^{2/3}} \leq 2.5C_a \qquad \text{(NEHRP eq. 3.1.3-7)}$$

C_v = **Seismic coefficient**, expressed as a fraction of acceleration of gravity and determined based on **Soil profile type** (A to E) and the **effective peak velocity-related acceleration** A_v (may be obtained from NEHRP map 2). $_v C$ may be obtained in accordance with the 1994 NEHRP section 1.4.2.3 or from Table D.2 in appendix D (1994 NEHRP Table 1.4.2.4b). The maximum value of C_v is 0.96 (for Soil Profile Type E).

T = **Effective fundamental period** of the structure.

x = **Elevation of nonstructural component** relative to grade elevation.

h = **Average roof elevation** of structure relative to grade elevation.

3.4.2 *Seismic Displacement Requirement*

Seismic relative displacement D_p between two connection points on the same building or structural system shall be designed to accommodate the smaller of the following two equations (1994 NEHRP section 3.1.4):

$$D_p = \delta_{xA} - \delta_{yA} \qquad \text{(NEHRP equation 3.1.4-1)}$$

or

$$D_p = (X - Y)\Delta_{aA}/h_{sx} \qquad \text{(NEHRP equation 3.1.4-2)}$$

For two connection points on separate buildings or structural systems (e.g. connection points across an expansion joint), D_p shall be designed to accommodate the smaller of the following equations:

$$D_p = |\delta_{xA}| + |\delta_{yB}| \qquad \text{(NEHRP equation 3.1.4-3)}$$

or

$$D_p = X\Delta_{aA}/h_{sx} + Y\Delta_{aB}/h_{sx} \qquad \text{(NEHRP equation 3.1.4-4)}$$

where:

δ_{xA} = Deflection at building level x of Building A, determined from elastic analysis and multiplied by the **deflection amplification factor** C_d. C_d values for different structural systems are listed in Table D.5 in appendix D (1994 NEHRP Table 2.2.2).

δ_{yA} = Deflection at building level y of Building A, determined from elastic analysis and multiplied by the C_d factor.

δ_{xB} = Deflection at building level x of Building B, determined from elastic analysis and multiplied by the C_d factor.

δ_{yB} = Deflection at building level y of Building B, determined from elastic analysis and multiplied by the C_d factor.

X = Height of upper support attachment at level x as measured from grade.

Y = Height of lower support attachment at level y as measured from grade.

Δ_{aA} = Allowable story drift for Building A as defined in Table D.6 in appendix D (1994 NEHRP Table 2.2.7).

Δ_{aB} = Allowable story drift for Building B as defined in Table D.6 in appendix D (1994 NEHRP Table 2.2.7).

h_{sx} = Story height used in the definition of the allowable drift, Δ_a, in Table D.6. Δ_a/h_{sx} is the allowable drift index.

The 1994 NEHRP Recommended Provisions also prescribe required clearances for suspended ceiling and other ceiling components such as fire sprinkler heads and light fixtures.

3.5 Seismic Building Codes of Japan[11,12]

Seismic design requirements for building mechanical equipment and nonstructural components are provided by two documents: (1) *Guideline for Aseismic Design and Construction of Building Equipment*, published by the Building Center of Japan (1984) and (2) *Guideline for Aseismic Design for Architectural Nonstructural Elements*, published by Public Building Association (1987). The coefficients used in determining the design seismic force are similar between these two guidelines when the *Modified Seismic Coefficient Method* is used as the design method. The requirements for nonstructural components and building equipment are described below.

3.5.1 *Seismic Force Requirement*

Nonstructural components and their attachments shall be designed to resist the total lateral design seismic force, F_H, which was prescribed based on the modified seismic force coefficients method:

$$F_H = K_H W$$

and,

$$K_H = ZIK_1K_2k_0$$

where:

K_H is the **lateral design seismic force coefficient.**

W is the **weight** of the nonstructural component or equipment (unit: kgf).

Z is the **seismic zone factor**. $Z = 1.0$ for Seismic Zone A, 0.85 for Seismic Zone B, and 0.70 for Seismic Zone C (see Figure E.1 in Appendix E).

I is the **seismic importance reduction factor**. $I = 1.0$ for important building equipment, and 2/3 for general building equipment. Building owners and structural designers can determine importance and select the appropriate value for I.

K_1 is the **floor response amplification factor** of a building, which varies between 1.0 and 3.33. $K_1 = 1.0$ at the basement floor and 3.33 at roof level.

K_2 is the **response amplification factor** of the nonstructural component or equipment. Specific values of K_2 are provided for some architectural components. At the time of this writing, this information was not available. In general, K_2 ranges between 1.0 and 2.0.

k_0 is the **standard design seismic force coefficient** (0.3).

The value of $I.K_1.K_2.k_0$ shall be not less than 0.6 for important nonstructural components and 0.3 for ordinary nonstructural components.

The vertical design seismic force, F_v, shall be determined by the following formula:

$$F_V = K_V W$$

where K_V is the **vertical design seismic force coefficient** $(K_V = K_H/2)$.

3.5.2 *Seismic Displacement Requirement*

Similar to other codes, seismic displacement requirements are not as specific as the seismic force requirements. The current building codes of Japan require that for pipes, vertical pipes shall be subjected to a maximum story drift of 1/200 radian times the story height. Pipes through expansion joints shall be designed for possible maximum relative displacement between two structures.

3.6 Summary

Provisions relevant to the seismic design requirements for nonstructural building components of five seismic engineering documents, which include the 1994 Uniform Building Code, the 1994 Standard Building Code, the 1994 NEHRP Recommended Provisions, the 1992 New Zealand Standard (NZS 4203), and the 1982 and 1987 Japanese codes, are reviewed and summarized in sections 3.1 to 3.5. The review shows wide variations in seismic design requirements between codes, both in terms of seismic force and displacement calculations and in listings of nonstructural components and corresponding coefficients.

For the *seismic force requirement*, the building codes and the NEHRP Recommended Provisions use three basic coefficients to account for the following factors in prescribing the design force:

- **Seismicity of the region** where the building is located (*seismic zone factor,* or *effective peak-velocity acceleration,* or *component acceleration coefficient*).

- **Functionality of nonstructural components and buildings** in terms of life-safety importance (*seismic importance factor*, or *component risk factor*, or *component performance criteria*).

- **Response characteristics of nonstructural components** to seismic lateral load (*component seismic coefficient*, or *component horizontal force factor*, or *component response amplification factor*).

The above three factors are considered in the seismic force requirements of the 1994 UBC, 1994 SBC, and in equation 3.1.3-1 of the 1994 NEHRP Recommended Provisions. Other factors not explicitly included in the seismic design requirements of the above three codes, but which are explicitly considered in the 1992 NZS 4203, the 1982 and 1987 Japanese building codes, and in equation 3.1.3-2 of the 1994 NEHRP Recommended Provisions are:

- **Response characteristics of the building** to seismic lateral load (*building seismic coefficient*).

- **Site soil profile** (*building seismic coefficient*).

- **Component location** relative to building height (*floor response amplification factor*).

Table 3.1 summarizes the coefficients affecting the calculation of seismic lateral force of the codes reviewed. In terms of level of detail, the seismic design requirements of the 1994 NEHRP Recommended Provisions (equation 3.1.3-2) and the 1992 NZS 4203 appear to require the most detailed information for the calculation of design lateral force for nonstructural components. Of all five documents reviewed, only the 1994 NEHRP Recommended Provisions provides two alternate methods for computing the seismic lateral force requirement.

For the *seismic displacement requirement*, the 1994 SBC and the 1994 NEHRP Recommended Provisions are more specific than other building codes in prescribing the required seismic lateral displacement. Both of these documents provide formulas for calculating the displacement at points of support for nonstructural components. The 1994 NEHRP Provisions also specifically prescribe detailed requirements for clearance between co-located ceiling components, such as clearance between suspended ceiling and fire sprinkler heads. The 1994 UBC, New Zealand Standard NZS 4203, and the 1982 and 1987 seismic building codes of Japan are less precise in prescribing seismic displacement requirements for nonstructural building components. In general, all codes require that attention be paid to the relative displacement between connection points of nonstructural components, especially connection points that are located on separate structural systems or buildings (anchors for piping systems crossing expansion joints, for example).

Table 3.1 **Coefficients Affecting Seismic Force Requirements in Various Building Codes**

Coefficients to account for ⇓	**1994 UBC**	**1994 SBC**	**1994 NEHRP Eq. 3.1.3-1**	**1994 NEHRP Eq. 3.1.3-2**	**NZS 4203**	**Japan**
1) Seismicity of Region Where Buildings are Located	• Seismic Zone Factor: Z (0.075 - 0.4)	• Effective Peak-Velocity Acceleration: A_v (0.05 - 0.4)	• Seismic Coefficient: C_a (0.04 - 0.44) (varies with soil type)	• Component Acceleration Coefficient: A_p ($\leq 4.0\ C_a$)	• Seismic Zone Factor: Z (0.4 - 0.8)	• Seismic Zone Factor: Z (0.7 - 1.0)
2) Functionality of Components in Terms of Life-Safety Importance	• Seismic Importance Factor: I_p (1.0 - 1.5) (varies with Occupancy Category)	• Performance Criteria Factor: P (0.5 - 1.5) (varies with Seismic Hazard Exposure Group)	• Component Importance Factor: I_p (1.0 - 1.5) (varies with degree of affect on life-safety)	• Component Importance Factor: I_p (1.0 - 1.5) (varies with degree of affect on life-safety)	• Component Risk Factor: R_p (1.0 - 1.1) (varies with degree of affect on life-safety). • Building Risk Factor: R (0.6 - 1.3)	• Seismic Importance Reduction Factor: I (1.0 or 2/3)

Table 3.1 (**continued**)

Coefficients to account for ⇓	**1994 UBC**	**1994 SBC**	**1994 NEHRP Eq. 3.1.3-1**	**1994 NEHRP Eq. 3.1.3-2**	**NZS 4203**	**Japan**
3) Component Seismic Response Characteristics	• Horizontal Force Factor: C_p (0.75 - 2.0)	• Component Seismic Coefficient: C_c (0.6 - 3.0) • Attachment Amplification Factor: a_c (1.0 - 2.0)	• Implied in constant: 4.0	• Component Amplification Factor: a_p (1.0 - 2.5) • Component Response Modification Factor: R_p (1.5 - 6.0)	• Implied in Seismic Coefficient: C_{ph} (through component ductility factor μ_p)	• Response Amplification Factor: K_2 (1.0 - 2.0)
4) Building Response Characteristics and Soil Site Profile	• Not explicitly considered	• Not explicitly considered	• Not explicitly considered	• Implied in Component Acceleration Coefficient: A_p	• Implied in Seismic Coefficient: C_{ph}	• Not explicitly considered
5) Component Location Relative to Building Height	• Not considered	• Not considered	• Not considered	• Implied in Component Acceleration Coefficient: A_p	• Implied in Seismic Coefficient: C_{ph}	• Floor Response Amplification Factor: K_1 (1.0 - 10/3) (varies with height)

4. COMPARISON OF CODE SEISMIC DESIGN REQUIREMENTS

4.1 Introduction

As can be seen in Chapter 3, there are noticeable variations in code requirements for nonstructural building components, both in terms of level of detail in the requirements and in the calculation procedures. Some codes have more detailed descriptions of nonstructural building components and assign more specific coefficients to various components, while others are less specific in listing the applicable components. In such cases the seismic coefficients necessary for computing seismic lateral force and displacement requirements can only be estimated.

In the following sections, comparison of cases where maximum seismic forces are required by the codes reviewed in chapter 3 will be conducted. The difference in seismicity in different countries is accounted for by using the maximum local seismic zone factors for the appropriate countries. Comparison of seismic displacement requirement also will be discussed. In addition, comparisons of seismic force requirements for two example problems for various nonstructural building components are performed to further illustrate the variation in seismic requirements of the major building codes.

4.2 Comparison of Maximum Seismic Force Requirement

Table 4.1 summarizes the seismic lateral force requirements and the conditions which result in maximum seismic force requirements for various nonstructural components by the four building codes and the 1994 NEHRP Recommended Provisions. The components listed in Table 4.1 are selected from the 1994 SBC list of nonstructural components, since this code appears to have the most detailed list and description of the components. For uniformity, different terminologies between codes which refer to the same quantity are made consistent in Table 4.1. For example, W_p is used for all codes in Table 4.1 to refer to the weight of nonstructural components, instead of W_c as used in the 1994 SBC. The 1994 NEHRP Recommended Provisions provide two different methods for computing the seismic lateral force requirement for nonstructural building components. One is given in NEHRP equation 3.1.3-1 which does not consider the component's amplification and response modification factors, while the other, given in NEHRP equation 3.1.3-2, considers these factors. Thus, two columns which list seismic force requirements according to the two methods of 1994 NEHRP Recommended Provisions are provided in Table 4.1.

As seen from Table 4.1, significant variations in maximum lateral force requirements exist between codes. In terms of *maximum seismic force requirements*, the 1994 NEHRP Recommended Provisions equation 3.1.3-1 appears to be most stringent. Next are the 1992 New Zealand Standard NZS 4203 and the Japanese building codes (1982 and 1987). The 1994 SBC is as stringent as NEHRP equation 3.1.3-1 and even more stringent than the NZS 4203 and the Japanese codes when dealing with safety equipment such as fire protection equipment and pipe systems. For other components, the 1994 SBC seismic force requirements are in general less conservative than those of the above codes. The 1994 UBC appears to be the least stringent of all building codes and recommended provisions reviewed. The difference in level of seismic force requirement between the codes can be more than five times for some components. For example, maximum seismic force requirements for fire protection equipment and system vary from $0.45W_p$ to $2.64W_p$ between the 1994 UBC and equation 3.1.3-1 of the 1994 NEHRP Recommended Provisions.

Table 4.1 **Comparison of maximum seismic force requirement of various codes**

CODES	1994 UBC	1994 SBC	NZS 4203 (1992)	1994 NEHRP		1987 JAPAN
CONDITIONS FOR MAXIMUM SEISMIC FORCE REQUIREMENTS	• Structures are located in Seismic Zone 4 (Z=0.4). • Structures are Essential & Hazardous facilities, or Standard Occupancy Structures with anchorage of machinery and equipment for life-safety systems (I_p=1.5).	• Structures are located in regions where Effective Peak Velocity-related Acceleration Coefficient, A_v = 0.4. • Structures are of Seismic Hazard Exposure Group III. • Resilient mounting systems are of the types which cause Attachment Amplification Factor a_c to be 2.0.	• Structures are located in regions of maximum Zone Factor (Z=0.8). • Components and attachments are designated as P.I and P.II (R_p = 1.1).	• Structures are located at sites with Soil Profile Type D and Effective Peak Ground Acceleration Coefficient A_a = 0.4 (C_c = 0.44). • Structures are located in regions where Effective Peak Ground Acceleration Coefficient A_a = 0.4.		• Structures are located in seismic zone A (Z = 1.0). • Equipment/ components are designated as Important Building Equipment. • Equipment/ components are located at top level of building (K_l = 10/3).
1. Exterior nonbearing walls	$0.45W_p$	$0.54W_p$	$0.60W_p$	$1.76W_p$[d]	$0.88W_p$[e]	$1.33W_p$
2. Wall attachments	$1.2W_p$	$1.8W_p$	$2.2W_p$	$1.76W_p$[d]	$0.88W_p$[e]	$1.33W_p$
3. Veneer connections	$1.2W_p$	$1.2W_p$	$2.2W_p$	$1.76W_p$[d]	$1.76W_p$[e]	$1.33W_p$
4. Fire protection equipment and systems	$0.45W_p$	$2.4W_p$	$2.2W_p$[a]	$2.64W_p$[d]	$1.65W_p$[e]	$2.0W_p$
5. Pipe systems	$0.45W_p$	$2.4W_p$[b]	$2.2W_p$[a]	$2.64W_p$[d]	$1.65W_p$[e]	$2.0W_p$
6. Suspended ceilings	$0.45W_p$	$0.54W_p$[c]	$2.0W_p$[a]	$2.64W_p$[d]	$1.76W_p$[e]	$1.33W_p$
7. Lighting fixtures	$0.45W_p$	$0.804W_p$	$2.0W_p$[a]	$1.76W_p$[d]	$1.76W_p$[e]	$1.33W_p$

Notes: [a] Computed with μ_p assumed to equal 1.0 since values of μ_p for these components are not available in the 1992 NZS 4203.
[b] Pipe systems for gas and high hazard piping.
[c] For fire-rated membrane.
[d] Computed using 1994 NEHRP equation 3.1.3-1, $F_p = 4.0C_aI_pW_p$.
[e] Computed using 1994 NEHRP equation 3.1.3-2, $F_p = a_pA_pI_pW_p/R_p$

4.3 Comparison of Seismic Displacement Requirement

Numerical comparison of code-prescribed seismic displacement requirements is not possible since the calculation for displacement of nonstructural building components requires case-specific information such as building mode of vibration, modal period, and base shear, etc. Thus comparisons similar to section 4.2 are not conducted here. Instead, only a general discussion is presented here for comparative purposes.

The 1994 NEHRP Recommended Provisions provide the most detailed seismic displacement requirements compared to other building codes reviewed. Besides formulas prescribing the relative seismic displacement between connection points for nonstructural components on the same building (NEHRP equations 3.1.4-1 and 3.1.4-2) and on separate buildings (NEHRP equations 3.1.4-3 and 3.1.4-4), NEHRP Provisions also prescribe clearances for co-locating systems such as suspended ceiling and fire sprinkler heads.

The 1994 SBC provides specific formulae for computing seismic displacement for nonstructural components. SBC's *architectural components* are required to accommodate design story drift, which is computed as the difference between story-level displacements. In computing story-level displacement, SBC considers *deflection amplification* of different seismic resisting systems.

Seismic displacement requirements of other codes besides the 1994 NEHRP and the 1994 SBC are much less specific. In general, all codes require that differences in elevations and in structural systems between connection points shall be considered in computing seismic displacement of connection points.

4.4 Comparison of Force Requirements for Example Case Problems

Section 4.2 above provides a comparison for *maximum seismic force requirements*, which correspond to extreme situations involving *life-safety* components or equipment, in *critical* facilities located *in the most active seismic* regions. This comparison, summarized in Table 4.1, revealed wide variation in the maximum seismic force requirements prescribed by the reviewed seismic codes. To further illustrate the variations in seismic requirements between codes, two example case problems are shown in the following sections. Example case problems are selected for intermediate and noncritical cases, i.e. situations which would not result in maximum seismic force requirements, so that their comparisons can be used together with the maximum situation to better illustrate the variation in codes.

4.4.1 *Example Case Problem 1*

Compute design seismic force requirements for the following nonstructural building components:

- *Nonfire-rated suspended ceiling*
- *Partitions, nonbearing walls*
- *Fire sprinkler system*

The components are located on the top floor of a five-story concrete frame building which is a hospital with surgery and an emergency treatment area. The hospital is located in the Midwest (St. Louis, Missouri) and sited on rocky and stiff soil.

UBC 1994:

• $Z = 0.15$	(St. Louis, MO. ⇒ UBC seismic zone 2A)
• $I_p = 1.5$	(Essential Facility - Hospital with surgery and emergency treatment area)
• $C_p = 0.75$	For suspended ceiling (UBC does not have a specific C_p value for suspended ceilings. Therefore, C_p value for anchorage for ceilings is used).
$= 0.75$	For partition walls.
$= 0.75$	For fire sprinkler system.

⇒ $F_p = 0.17W_p$ for **suspended ceiling**
$= 0.17W_p$ for **partition wall**
$= 0.17W_p$ for **fire sprinkler system**

SBC 1994:

• $A_v = 0.2$	(from SBC seismic zone map for St. Louis, MO.)	
• $P = 1.0$,	$C_c = 0.6$	For suspended ceiling (Seismic Hazard Exposure Group III)
$= 1.5$,	$= 0.9$	For interior nonbearing wall (SHEG III)
$= 1.5$,	$= 2.0$	For fire sprinkler system (SHEG III)
• $a_c = 1.0$	(fire sprinkler system with fixed or direct connection).	

⇒ $F_p = 0.12W_p$ for **suspended ceiling**
$= 0.27W_p$ for **partition wall**
$= 0.60W_p$ for **fire sprinkler system**

NEHRP 1994:

• $I_p = 1.0$	For suspended ceiling
$= 1.5$	For interior nonbearing wall which can block means of egress when failed
$= 1.5$	For fire sprinkler system which is required to function after an earthquake

If Equation 3.1.3-1 ($F_p = 4.0C_aI_pW_p$) is used:

• $C_a = 0.16$ since $A_a = 0.10$ (From NEHRP map 1, building is in St.Louis, MO) and Soil Type D (stiff soil).

⇒ $F_p = 0.64W_p$ for **suspended ceiling**
$= 0.96W_p$ for **partition wall**
$= 0.96W_p$ for **fire sprinkler system**

If Equation 3.1.3-2 ($F_p = a_p A_p I_p W_p / R_p$) is used:

Fundamental period T for five story concrete building:

$T = 0.1N = 0.1(5) = 0.5$ second
$C_v = 0.24$ since $A_v = 0.1$ (from NEHRP map 2) and Soil Type D.

$A_s = 1.2C_v/T^{2/3} = 1.2(0.24)/0.5^{2/3} = 0.46$ (> $2.5C_a = 0.4$) $\Rightarrow$Use $A_s = 0.4$
$A_r = 2.0A_s = 2.0(0.4) = 0.80$ (> $4.0C_a = 0.64$) $\Rightarrow$Use $A_r = 0.64$

Components located on top floor, $x = h$, $\Rightarrow x/h = 1.0$

- $A_p = C_a + (A_r - C_a)(x/h) = 0.16 + (0.64 - 0.16)(1.0) = 0.64$
- $a_p = 1.0$, $R_p = 1.5$ For suspended ceiling
  $= 1.0$, $= 3.0$ For partition nonbearing walls
  $= 2.5$, $= 4.0$ For fire sprinkler system

$\Rightarrow$ $F_p = 0.43W_p$ for **suspended ceiling**
$= 0.32W_p$ for **partition wall**
$= 0.60W_p$ for **fire sprinkler system**

Table 4.2 **Summary of Seismic Force Requirements for Example Problem 1**

Nonstructural Components	**1994 UBC**	**1994 SBC**	**1994 NEHRP Recommended Provisions**	
			Eq. 3.1.3-1	**Eq. 3.1.3-2**
Suspended Ceiling	$0.17W_p$	$0.12W_p$	$0.64W_p$	$0.43W_p$
Partition Wall	$0.17W_p$	$0.27W_p$	$0.96W_p$	$0.32W_p$
Fire Sprinkler System	$0.17W_p$	$0.60W_p$	$0.96W_p$	$0.60W_p$

4.4.2 *Example Case Problem 2*

Compute the design seismic force requirements for:

- *Nonfire-rated suspended ceiling*
- *Partitions, nonbearing walls*
- *Fire sprinkler system*

The components are located on the first floor of a 3-story concrete frame commercial office building, located in the Northeastern U.S. The building is sited on stiff soil as in example problem 1.

<u>UBC 1994</u>:

- Z = 0.15 (UBC seismic zone 2A)
- I_p = 1.0 (Standard occupancy structures)
- C_p = 0.75 For suspended ceiling
  = 0.75 For partition walls.
  = 0.75 For fire sprinkler system.

$\Rightarrow$ $F_p = 0.11W_p$ for **suspended ceiling**
$= 0.11W_p$ for **partition wall**
$= 0.11W_p$ for **fire sprinkler system**

<u>SBC 1994</u>:

- $A_v = 0.1$
- P = 0.5, C_c = 0.6 For suspended ceiling (Seismic Hazard Exposure Group I)
  = 1.0, = 0.9 For interior nonbearing wall (SHEG I)
  = 1.5, = 2.0 For fire sprinkler system (SHEG I)
- a_c = 1.0 (fire sprinkler system with fixed or direct connection).

$\Rightarrow$ F_p = Exempted for **suspended ceiling** (according to 1994 SBC section 1607.6 for components with SHEG I, P=0.5,A_v=0.1)
$= 0.09W_p$ for **partition wall**
$= 0.30W_p$ for **fire sprinkler system**

<u>NEHRP 1994</u>:

- I_p = 1.0 For suspended ceiling
  = 1.0 Partition wall which does not block means of egress when failed
  = 1.5 For fire sprinkler system which is required to function after an earthquake

If Equation 3.1.3-1 (F_p = *$4.0C_aI_pW_p$*) is used:

- C_a = 0.16 since A_a = 0.10 (From NEHRP map 1 for Northeast region) and Soil Type D

$\Rightarrow$ $F_p =$ $0.64W_p$ for **suspended ceiling**
$=$ $0.64W_p$ for **partition wall**
$=$ $0.96W_p$ for **fire sprinkler system**

If Equation 3.1.3-2 ($F_p = a_p A_p I_p W_p / R_p$) is used:

Fundamental period T for 3-story concrete building:

$T = 0.1N = 0.1(3) = 0.3$ second
$C_v = 0.24$ since $A_v = 0.1$ (from NEHRP map 2) and Soil Type D.

$A_s = 1.2C_v/T^{2/3} = 1.2(0.24)/0.3^{2/3} = 0.64$ ($> 2.5C_a = 0.4$) $\Rightarrow$Use $A_s = 0.4$
$A_r = 2.0A_s = 2.0(0.4) = 0.80$ ($> 4.0C_a = 0.64$) $\Rightarrow$Use $A_r = 0.64$

Components located on first floor of a 3-story building, $\Rightarrow x/h = 1/3 = 0.33$

- $A_p = C_a + (A_r - C_a)(x/h) = 0.16 + (0.64 - 0.16)(0.33) = 0.32$
- $a_p = 1.0$, $R_p = 1.5$ For suspended ceiling
  $= 1.0$, $= 3.0$ For partition nonbearing walls
  $= 2.5$, $= 4.0$ For fire sprinkler system

$\Rightarrow$ $F_p =$ $0.21W_p$ for **suspended ceiling**
$=$ $0.10W_p$ for **partition wall**
$=$ $0.30W_p$ for **fire sprinkler system**

Table 4.3 **Summary of Seismic Force Requirements for Example Problem 2**

Nonstructural Components	**1994 UBC**	**1994 SBC**	**1994 NEHRP Recommended Provisions**	
			Eq. 3.1.3-1	**Eq. 3.1.3-2**
Suspended Ceiling	$0.11W_p$	Exempted	$0.64W_p$	$0.21W_p$
Partition Wall	$0.11W_p$	$0.09W_p$	$0.64W_p$	$0.10W_p$
Fire Sprinkler System	$0.11W_p$	$0.30W_p$	$0.96W_p$	$0.30W_p$

4.5 Summary

Seismic design requirements for various nonstructural building components by four building codes, including the 1994 UBC, 1994 SBC, 1992 NZS 4203, the 1984 and 1987 Japan's building codes, and the 1994 NEHRP Recommended Provisions for Seismic Regulation for New Buildings were compared. For *seismic force requirements*, the comparisons included three cases:

- *Maximum seismic force requirement*, which corresponds to the most critical situations prescribed by the codes (life-safety equipment in buildings categorized as essential or hazardous and located in the most active seismic region).

- *Moderate seismic force requirement*, which corresponds to a less critical situation than above (nonlife-safety components in essential buildings, located in a moderate seismic region).

- *Low seismic force requirement*, which corresponds to the least critical situation (nonlife-safety components in non-critical buildings, located in a moderate seismic region).

The results of the comparisons are shown in Tables 4.1, 4.2, and 4.3. In terms of maximum seismic force requirement, NEHRP (1994) equation 3.1.3-1 appears to be the most conservative. This is followed by the 1992 New Zealand Code NZS 4203, then the NEHRP (1994) equation 3.1.3-2 and the Japanese Codes (1982 and 1987). Next is the 1994 SBC. The least stringent is the 1994 UBC, as shown in Table 4.1. The difference between the most and least stringent requirements can be more than five times as shown in Table 4.1 for the cases of *fire protection equipment and systems*, *pipe system*, and *suspended ceilings* computed by the 1994 NEHRP equation 3.1.3-1 and the 1994 UBC ($2.64W_p$ and $0.45W_p$, respectively).

For the example case problems, comparisons included only the building codes used in the U.S. (UBC and SBC) and the NEHRP Recommended Provisions. The New Zealand and Japanese codes were excluded due to difference in seismic regions. The same trend was observed as shown in Tables 4.2 and 4.3, with the 1994 NEHRP equation 3.1.3-1 being the most stringent, followed by the 1994 NEHRP equation 3.1.3-2, then the 1994 SBC, and lastly the 1994 UBC.

The comparisons of seismic force requirements also revealed the lack of flexibility in assigning different levels of importance to different nonstructural components of the 1994 UBC. This is evidenced by the same level of force requirements for various components, even though these components have different levels of importance. For example, both the 1994 SBC and the NEHRP Recommended Provisions require higher design lateral forces for *fire sprinkler systems*, which are considered more important in terms of life-safety than *suspended ceilings*, while the 1994 UBC consistently prescribed the same design force level for both of these components. Furthermore, the 1994 SBC exempts the seismic force requirement for components with *performance criteria factor* $P=0.5$, located in structures with *seismic performance category* C ($A_v=0.1$) and with *seismic hazard exposure group* I. No such exemption is allowed in other codes.

For *seismic displacement requirements*, a similar comparison was not conducted due to the detailed information required for calculation of displacement, such as the building mode of vibration, modal period, base shear, and the portion of base shear to be attributed to the component's elevation. Thus, only general discussions were provided. Of the four building codes and the NEHRP Recommended Provisions reviewed, only the NEHRP Provisions and the SBC prescribe specific formulas for computing relative seismic displacements at the connection points of nonstructural components. The 1994 NEHRP also prescribes clearances for various co-located systems such as suspended ceilings and fire sprinkler heads, etc. Other codes require in general that differences in elevation and in structural systems (separated by expansion joints, for example) between connection points be accommodated in computing seismic displacements for nonstructural building components.

5. SUMMARY, DISCUSSIONS & RECOMMENDATIONS

5.1 Summary

This report is the first of the current effort at the National Institute of Standards and Technology (NIST) focusing on the subject of mitigation of seismic damage to nonstructural components, especially ceiling-located components. The report summarizes recently observed damage to ceiling components, and reviews and compares relevant seismic design requirements of various major building codes.

Widespread damage sustained by nonstructural components, especially components related to ceilings, during recent earthquakes has illustrated the continuing need for evaluation of existing seismic design requirements and for development of methods to mitigate losses caused by damage to these nonstructural components. Despite the widespread damage reported in post earthquake surveys, only a handful of studies focusing on nonstructural performance has been identified.

There are wide variations in seismic design requirements for nonstructural building components between the two current U.S. national model building codes (the 1994 UBC, which is adopted in part by much of the western U.S., and the 1994 SBC, which is adopted in part by the southeastern U.S.), the 1992 New Zealand Standard NZS 4203, the current building codes of Japan (1982 and 1987), and the 1994 NEHRP Recommended Provisions for Seismic Regulations for New Buildings.

In terms of levels of detail, the 1994 UBC, 1994 SBC and 1994 NEHRP equation 3.1.3-1 appear to be simplest, with three factors considered for calculation of the seismic force requirement: (1) factor to account for *seismicity of regions*, (2) factor to account for *functionality of nonstructural components and buildings*, and (3) factor to account for *response characteristics of nonstructural components*. While the 1994 NEHRP equation 3.1.3-2, the 1992 NZS 4203, and the Japanese building codes require, in addition to the above three factors, more detailed information such as the *seismic response characteristics of the building*, *site soil profile* information, and *component location* relative to building height. Also, the 1994 SBC and the 1994 NEHRP Provisions are more specific in prescribing the seismic displacement requirements and clearances for nonstructural components than other building codes reviewed in this report.

In terms of levels of stringency, the variation between the most and least stringent seismic force requirement can be more than five times. The most stringent seismic force requirement is that of the 1994 NEHRP document. Next in stringency is the 1992 New Zealand Standard NZS 4203, and then the Japanese building codes. The least conservative code with respect to seismic force requirements for nonstructural building components is the 1994 UBC. The UBC also appears to lack the flexibility in assigning different levels of importance to different nonstructural components.

5.2 Discussions & Recommendations

The 1994 editions of the U.S. national model buildings codes and the NEHRP Recommended Provisions for Seismic Regulation for New Buildings contain the newest recommendations for seismic design requirements for nonstructural building components. These recommendations are more stringent than previous provisions, and are prescribed as a result of lessons learned from recent earthquakes. However, the effectiveness of the new recommendations in limiting damage to nonstructural components remains to be seen, since there currently are not enough new buildings affected by the new design provisions. Further, since the implementation of the new provisions and recommendations (1994), there has not been a major strong motion event in the U.S. to allow a thorough assessment. Thus, the need to mitigate nonstructural damage similar to that observed in past earthquakes still exists, especially for those buildings which were designed in accordance with older version of the building codes.

The variation in seismic force requirements between codes shown in this report, while appearing to be inconsistent at the very least and should be addressed by code writing organizations, is not necessarily the main reason contributing to the widespread damage to nonstructural components in past earthquakes. Rather, especially when ceilings components such as lighting fixtures, suspended ceilings, fire sprinkler systems, piping systems, and HVAC ductwork are concerned, displacement incompatibility might be at least equally, if not more, the cause of the damage. The problem might not be entirely a structural engineering problem, but also may stem from the difficulty of determining which discipline, among engineers, architects, and the various building trades, is ultimately responsible for the design and installation of nonstructural components. Installation of ceiling components is usually performed in accordance with existing Standard Practices, published by appropriate industries. It is unclear, however, if these Standard Practices reflect recent changes made in the building codes. For example, the 1994 NEHRP Recommended Provisions recognize Industry Standard Practices such as CISCA for suspended ceilings but insist that seismic forces be determined by the NEHRP Recommended Provisions (section 3.1.3 Seismic Force Requirements).

The wide gap in seismic design requirements between the most recent NEHRP Recommended Provisions (1994) and the national model codes is not surprising. This gap is even wider when the NEHRP Recommended Provisions are compared with state and local building codes. In the process of code development, NEHRP provisions often contain the earliest and highest level of requirements since the Recommended Provisions are usually the first seismic engineering document to incorporate seismic engineering research results. For these recommended provisions to be accepted and eventually adopted, consideration must be given not only to the engineering aspects of the recommended provisions, but also to the provision's cost-effectiveness and political acceptability. Based on the latest version of the NEHRP Recommended Provisions, national code writing organizations such as the International Conference of Building Officials (ICBO) and the Southern Building Code Congress International (SBCCI) review their corresponding provisions for possible revision or adoption. Thus, the NEHRP Recommended Provisions are usually not reflected in the national model codes until a few years after publication. This is why the 1994 UBC and SBC contain selected provisions of the NEHRP Recommended Provisions published prior to the current version (1994). A similar process takes place for the adoption of provisions from the national model codes into the state or local building codes. This results in significant differences in seismic requirements between the 1994 NEHRP, the 1994 UBC and SBC, and the state and local codes. The undesirable effect of this lengthy adoption process is that it tends to make codes minimal, rather than optimal requirements (so that the provisions have more chances to be adopted). Also, the independent selection of adoptable provisions by code writing organizations results in variation in design requirements for the same components.

A scarcity of focused studies, dedicated to mitigating seismic damage to nonstructural building components is indicative of the lack of attention paid to nonstructural damage to date, even though widespread damage to nonstructural building components continues to be observed in recent earthquakes. Only two experimental programs dealing with seismic performance of suspended ceilings have been identified. The latest study was conducted in 1984. No program which addresses the problem of displacement incompatibility of co-located ceiling components, which was a principal cause of damage to these components in the 1994 Northridge earthquake, has been conducted to date.

Based on these findings, three major areas of needed research are identified. A focused research program, expected to have both experimental and analytical phases is planned at NIST to address the following issues:

***A. Issues related to building codes*:**

A.1 Validate, by experiments, the adequacy of the seismic force and displacement requirements of current seismic engineering codes, and establish threshold levels for acceptable performance of ceiling-located components.

A.2 Address variations in the code-prescribed seismic lateral design force and displacement requirements, based on results of experimental studies, in cooperation with code writing organizations.

***B. Issues related to mitigation techniques*:**

Develop techniques to mitigate damage to ceiling components designed in accordance with older codes, since there is a significant stock of existing buildings that were built in accordance with older codes. The seismic vulnerability of nonstructural components in these buildings was demonstrated in recent earthquakes as described in section 2.1. Potential techniques should be validated by an experimental program which includes dynamic testing. The techniques to be developed should address the following:

B.1 Lateral displacement compatibility between co-located ceiling components: fire sprinkler heads, suspended ceilings, lighting fixtures, and HVAC ducts.

B.2 Vertical displacement compatibility between ceiling components.

C. *Issues related to a common design guidelines for ceiling components*

C.1 Uniform guidelines for design, installation, and seismic restraint methods for ceiling components, incorporating new research results, new code-prescribed seismic requirements, and existing Standard Practices used by industries, are desirable. The guidelines to be developed should address not only the needs of structural engineers, but also the needs of other disciplines such as architects, manufacturers, and the building trades.

6. APPENDICES

APPENDIX A. Tables Related to UBC 1994

Table A.1 **UBC 1994 Seismic Zone Factor *Z***
(UBC 1994 Table 16-I)

Zone	**1**	**2A**	**2B**	**3**	**4**
Z	0.075	0.15	0.20	0.30	0.40

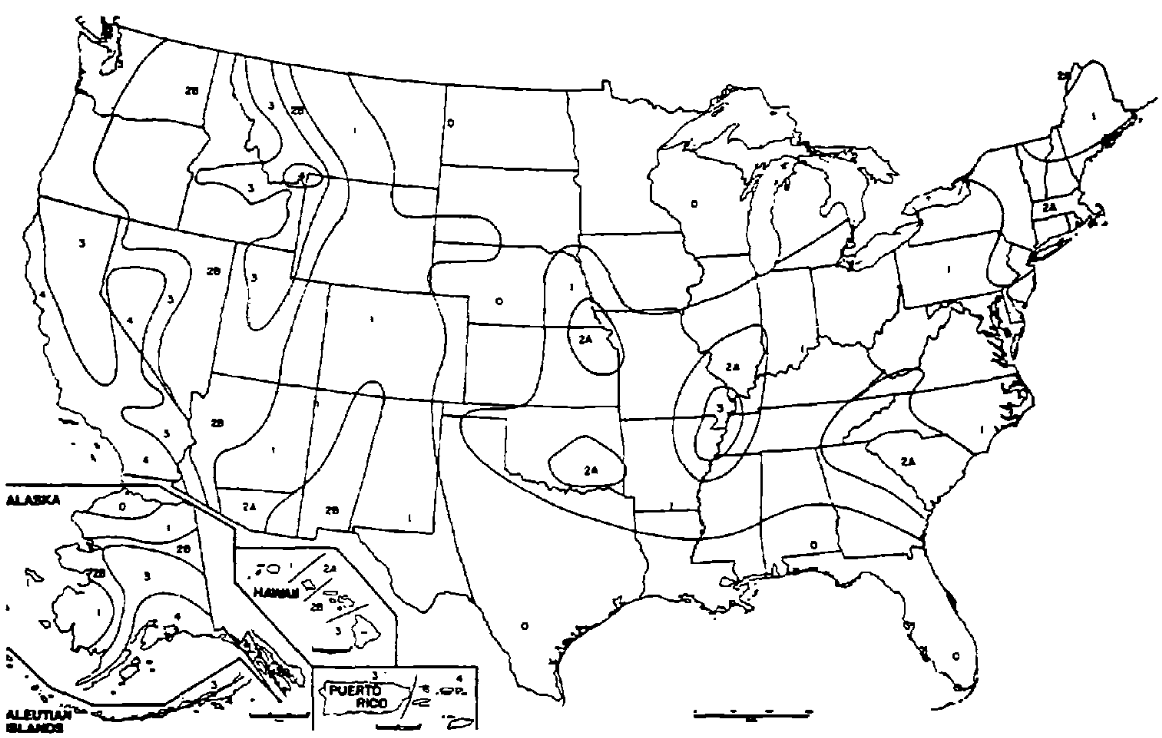

Figure A.1 **UBC 1994 Seismic Zone Map of the United States**
(UBC 1994 Figure 16-2)

Table A.2 **Seismic Importance Factor I_p**
(From UBC 1994 Table 16-K)

Occupancy Category	**Occupancy or Functions of Structure**	**Seismic Importance Factor, I_p**
1. Essential Facilities	•Group I, Division 1 Occupancies having surgery and emergency treatment areas. •Fire and police station. •Garages and shelters for emergency vehicles and emergency aircraft. •Structures and shelters in emergency-preparedness centers. •Aviation control towers. •Structures and equipment in government communication centers and other facilities required for emergency response. •Standby power-generating equipment for Category 1 facilities. •Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2 or 3 structures.	1.50
2. Hazardous facilities	• Group H, Divisions 1,2,6 and 7 Occupancies and structures therein housing or supporting toxic or explosive chemicals or substances. • Nonbuilding structures housing, supporting or containing quantities of toxic or explosive substances which, if contained within a building, would cause that building to be classified as a Group H, Division 1,2 or 7 Occupancy.	1.50
3. Special Occupancy Structures[1]	• Group A, Division 1,2 and 2.1 Occupancies. • Buildings housing Group E, Divisions 1 and 3 Occupancies with a capacity greater than 300 students. • Building housing Group B Occupancies used for college or adult education w/ a capacity greater than 500 students. • Group I, Divisions 1 and 2 Occupancies with 50 or more resident incapacitated patients, but not included in Category 1. • Group I, Division 3 Occupancies. • All structures with an occupancy greater than 5,000 persons. • Structures and equipment in power-generating stations; and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation.	1.00
4. Standard Occupancy Structures[1]	•All structures housing occupancies or having functions not listed in Category 1,2 or 3 and Group U Occupancy towers.	1.00
5. Miscellaneous Structures	• Group U Occupancies except for towers	1.00

[1] For anchorage of machinery and equipment required for life-safety systems the value of I_p shall be taken as 1.5.

Table A.3 **Horizontal Force Factor, C_p, for Nonstructural Components**
(From UBC 1994 Table 16-O)

Elements of Structures, Nonstructural Components and Equipment	C_p
Elements of Structures	
1. Walls including the following:	
a. Unbraced (cantilevered) parapets	2.0
b. Other exterior walls above the ground floor	0.75
c. All interior bearing and nonbearing walls and partitions	0.75
d. Masonry and concrete fences over 6 feet (1829 mm) high	0.75
2. Penthouse (except when framed by an extension of the structural frame)	0.75
3. Connections for prefabricated structural elements other than walls, with force applied at center of gravity	0.75
4. Diaphragms	-
Nonstructural Components	
1. Exterior and interior ornamentation and appendages.	2.00
2. Chimneys, stacks, trussed towers and tanks on legs:	
a. Supported on or projecting as an unbraced cantilever above the roof more than one half their total height.	2.00
b. All others, including those supported below the roof with unbraced projection above the roof less than one half its height, or braced or guyed to the structural frame at or above their centers of mass.	0.75
3. Signs and billboards.	2.00
4. Storage racks (include contents)	0.75
5. Anchorage for permanent floor-supported cabinets and book stacks more than 5 feet (1524 mm) in height (include contents).	0.75
6. Anchorage for suspended ceilings and light fixtures.	0.75
7. Access floor systems.	0.75
Equipment	
1. Tanks and vessels (include contents), including support systems and anchorage.	0.75
2. Electrical, mechanical and plumbing equipment and associated conduit, ductwork and piping, and machinery.	0.75

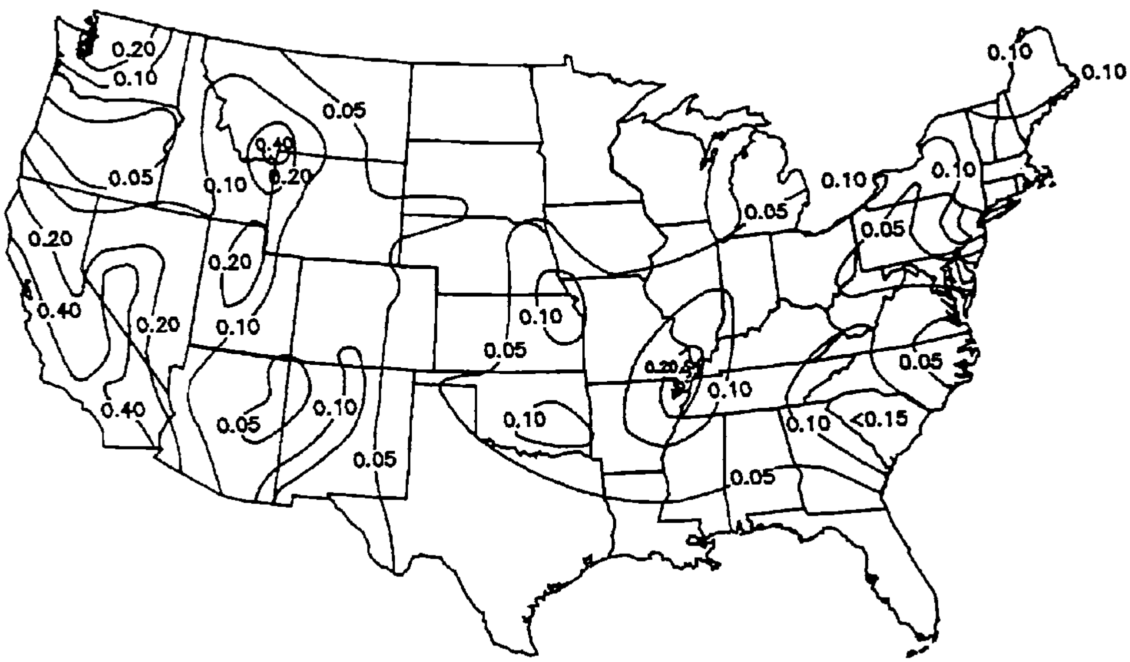

Figure B.1 **Contour Map of Effective Peak Velocity-Related Acceleration Coefficient, A_v** (SBC 1994 Figure 1607.1.5A).

Table B.1 **Architectural Component Seismic Coefficient (C_c) and Performance Criteria Factor (P)**

	Architectural Component	Component Seismic Coefficient (C_c)	Performance Criteria Factor (P) Seismic Hazard Exposure Group I	II	III
1.	Exterior nonbearing walls	0.9	1.5	1.5	1.5
2.	Interior nonbearing walls				
	Stair and elevator enclosures	1.5	1.0	1.0	1.5
	Other vertical shaft enclosures	0.9	1.0	1.0	1.5
	Other nonbearing walls	0.9	1.0	1.0	1.5
3.	Cantilever elements:				
	Parapets, chimney or stacks	3.0	1.5	1.5	1.5
4.	Wall attachments	3.0	1.5	1.5	1.5
5.	Veneer connections	3.0	0.5	1.0	1.0
6.	Penthouses	0.6	Not reqd.	1.0	1.0
7.	Membrane fire protection	0.9	1.0	1.0	1.5
8.	Ceilings				
	Fire-rated membrane	0.9	1.0	1.0	1.5
	Nonfire-rated membrane	0.6	0.5	1.0	1.0
9.	Storage racks more than 8 ft in height, contents included	1.5	1.0	1.0	1.5
10.	Access floors, supported equipment included	2.0	0.5	1.0	1.5
11.	Elevator & counterweight guardrails & supports	1.25	1.0	1.0	1.5

Table B.2 **Mechanical Electrical Component and System Seismic Coefficient (C_c) and Performance Criteria Factor (P)**

	Mechanical, Electrical Component or System	Component or System Seismic Coefficient (C_c)	Performance Criteria Factor (P) Seismic Hazard Exposure Group I	II	III
1.	Fire protection equipment and systems	2.0	1.5	1.5	1.5
2.	Emergency or standby electrical systems	2.0	1.5	1.5	1.5
3.	Elevator drive, suspension system and controller anchorage	1.25	1.0	1.0	1.5
4.	General Equipment A. Boilers, furnaces, incinerators, water heaters, and other equipment using combustible energy sources or high-temperature energy sources. B. Communication systems C. Electrical bus ducts and primary cable systems D. Electrical motor control centers, motor control devices, switchgear, transformers and unit substations. E. Reciprocating or rotating equipment F. Tanks, heat exchangers, and pressure vessels	2.0	0.5	1.0	1.5
5.	Manufacturing and process machinery	0.67	0.5	1.0	1.5
6.	Pipe systems				
	Gas and high hazard piping	2.0	1.5	1.5	1.5
	Fire suppression piping	2.0	1.5	1.5	1.5
	Other pipe systems	0.67	Not Reqd.	1.0	1.5
7.	HVAC ducts	0.67	Not Reqd.	1.0	1.5
8.	Electrical panel boards	0.67	Not Reqd.	1.0	1.5
9.	Lighting fixtures	0.67	0.5	1.0	1.5

Table B.3 **Seismic Hazard Exposure Group**
(1994 SBC Table 1607.1.6)

Group Type	**Nature of Occupancy**
Group I	All buildings except those listed below
Group II Seismic Hazard Exposure Group II buildings are those which have a substantial public hazard due to occupancy or use, including buildings containing any one or more of the indicated uses.	1. Group A in which more than 300 people congregate in one room. 2. Group E with an occupant load greater than 250. 3. Group B used for college or adult education with an occupant load greater than 500. 4. Group I Unrestrained with an occupant load greater than 50, not having surgery or emergency treatment facilities. 5. Group I Restrained. 6. Power generating stations and other public utility facilities not included in Group II Seismic Hazard Exposure Group. 7. Any other occupancy with an occupant load greater than 5,000.
Group III Seismic Hazard Exposure Group III buildings are those having essential facilities which are required for post earthquake recovery, including buildings containing any one or more of the indicated uses.	1. Fire or rescue and police stations. 2. Group I Unrestrained having surgery or emergency treatment facilities. 3. Earthquake emergency preparedness centers. 4. Postearthquake recovery vehicle garages. 5. Power generating stations and other utilities required as emergency backup facilities. 6. Primary communication facilities. 7. Highly toxic materials as defined by 308.2.1 (1994 SBC) as an H4 occupancy where the quantity of the material exceeds the exempt amounts of Table 308.2D (1994 SBC).

Table B.4 **Attachment Amplification Factor (a_c)**

Component mounting system (a_c)	**Attachment amplification factor**
Fixed or direct connection	1.0
Resilient mounting system	
Seismic activated restraining device	1.0
Elastic restraining device where:	
$T_c/T < 0.6$ or $T_c/T > 1.4$[1]	1.0
$T_c/T > 0.6$ or $T_c/T < 1.4$[1]	2.0

[1] T is the fundamental period of the building in seconds determined by 1994 SBC section 1607.4.1.2 or 1607.5.4. T_c is the fundamental period in seconds of the component and its attachment determined by 1994 SBC section 1607.6.4.1.

APPENDIX C. Tables Related to New Zealand Code

Table C.1 **Classification of Nonstructural Components and their Risk Factor R_p**
(NZS Tables 2.3.2 and 4.12.1)

Category	Description	R_p
P.I	Parts (nonstructural components and attachments), the failure of which could cause a life hazard.	1.10
P.II	Communications systems for which continuing function is important	1.10
P.III	Other parts	1.00

Table C.2 **Ductility Factors for Nonstructural Component**
(NZS 4203 Table C4.12.1)

Item	Description of Nonstructural Component	μ_p
1	Partitions, in-fill panels, prefabricated panels	
	(a) Connected so that instability is prevented if stiffness or strength degrades or if integrity is impaired	
	(i) Reinforced concrete or masonry	5.0
	(ii) Steel designed for ductility	6.0
	(iii) Timber or light-gauge steel framing	3.0
	(b) Other (e.g. vertical cantilevers)	
	(i) Reinforced concrete or masonry	
	Doubly reinforced	3.0
	Singly reinforced	2.0
	(ii) Steel or timber framing	1.0
2	Ornamentations, tied veneers, appendages	1.0
3	Floors and roofs acting as diaphragms, and other primary parts distributing seismic forces.	
	(a) Transfer diaphragms	1.0
	(b) Other	
	(i) Designed for limited ductility	3.0
	(ii) Otherwise	1.0

Table C.3 **Basic Seismic Acceleration Coefficient C_b**
(NZS 4203 Table 4.6.1 a, b, and c)

(a) Rock and very Stiff Soil Sites

Period *T or T_p* **seconds**	**Structural ductility factor μ**						
	1.0	1.25	2.0	3.0	4.0	5.0	6.0
0	0.40						
0.10	1.00						
0.15	1.00						
0.20	1.00						
0.25	0.94						
0.30	0.85						
0.35	0.77						
0.40	0.70	0.60	0.41	0.34	0.24	0.20	0.17
0.50	0.60	0.51	0.35	0.29	0.20	0.17	0.14
0.60	0.50	0.45	0.30	0.25	0.17	0.14	0.11
0.70	0.43	0.38	0.26	0.22	0.15	0.12	0.09
0.80	0.38	0.33	0.22	0.19	0.13	0.10	0.08
0.90	0.33	0.29	0.19	0.16	0.12	0.09	0.07
1.00	0.30	0.25	0.16	0.14	0.10	0.08	0.06
1.5	0.20	0.17	0.11	0.08	0.07	0.05	0.04
2.0	0.15	0.12	0.07	0.05	0.04	0.03	0.02
2.5	0.12	0.10	0.06	0.04	0.03	0.02	0.02
3.0	0.10	0.08	0.05	0.03	0.02	0.02	0.01
4.0	0.075	0.06	0.03	0.02	0.02	0.01	0.01

Table C.3 **Basic Seismic Acceleration Coefficient C_b**

(b) Intermediate soil sites

Period T or T_p seconds	Structural ductility factor μ						
	1.0	1.25	2.0	3.0	4.0	5.0	6.0
0	0.40						
0.10	0.80						
0.15	1.00						
0.20	1.00						
0.25	0.96						
0.30	0.89						
0.35	0.84						
0.40	0.80	0.69	0.47	0.38	0.28		0.20
0.50	0.73	0.63	0.43	0.34	0.25		0.17
0.60	0.67	0.58	0.39	0.31	0.22		0.14
0.70	0.62	0.53	0.35	0.27	0.19		0.12
0.80	0.57	0.49	0.32	0.25	0.17		0.107
0.90	0.53	0.45	0.29	0.22	0.15		0.095
1.00	0.50	0.42	0.26	0.20	0.13		0.087
1.5	0.33	0.27	0.17	0.127	0.08		0.056
2.0	0.25	0.20	0.13	0.097	0.06		0.042
2.5	0.20	0.16	0.10	0.075	0.05		0.033
3.0	0.17	0.13	0.085	0.064	0.04		0.028
4.0	0.125	0.10	0.063	0.047	0.03		0.021

Table C.3 **Basic Seismic Acceleration Coefficient C_b**

© Flexible or Deep Soil Sites

Period T or T_p seconds	Structural ductility factor μ						
	1.0	1.25	2.0	3.0	4.0	5.0	6.0
0	0.40						
0.10	0.80						
0.15	0.80						
0.20	0.80						
0.25	0.80						
0.30	0.80						
0.35	0.80						
0.40	0.80						
0.50	0.80						
0.60	0.80						
0.70	0.80						
0.80	0.80						
0.90	0.80	0.67	0.47	0.38	0.28	0.23	0.19
1.00	0.75	0.63	0.44	0.36	0.26	0.21	0.18
1.5	0.500	0.43	0.28	0.20	0.14	0.11	0.09
2.0	0.375	0.31	0.19	0.12	0.09	0.08	0.06
2.5	0.300	0.24	0.14	0.10	0.07	0.06	0.04
3.0	0.250	0.20	0.12	0.08	0.06	0.05	0.03
4.0	0.1875	0.15	0.09	0.06	0.04	0.03	0.02

Table C.4 **Building Classification and associated Risk Factor R**
(NZS Tables 2.3.1 and 4.6.3)

Category	Description	Risk Factor R
I	Building dedicated to the preservation of human life or for which the loss of function would have a severe impact on society.	1.3
II	Buildings which as a whole contain people in crowds.	1.2
III	Publicly owned buildings which house contents of a high value to the community.	1.1
IV	Buildings not included in any other category.	1.0
V	Buildings of a secondary nature.	0.6

Table C.5 **Values of $C_{ph}/(RZL_s)$ or $C_{ph}/(RZL_u)$ for Nonstructural Components with Varying Structural Ductility Factors, μ_p**
(NZS 4203 Table C4.12.2)

i	0	1	2	3	4	5	6	7	8	9	10
n						$\mu_p = 1$					
1	0.50	1.34									
2	0.50	1.18	1.86								
3	0.50	1.06	1.61	2.17							
4	0.50	0.98	1.45	1.93	2.40						
5	0.50	0.92	1.34	1.75	2.17	2.59					
6	0.50	0.88	1.25	1.63	2.00	2.38	2.76				
7	0.50	0.82	1.14	1.46	1.78	2.10	2.41	2.73			
8	0.50	0.77	1.05	1.32	1.59	1.86	2.14	2.41	2.68		
9	0.50	0.73	0.96	1.19	1.42	1.65	1.89	2.12	2.35	2.58	
10	0.05	0.70	0.90	1.10	1.29	1.49	1.69	1.89	2.09	2.29	2.49
n						$\mu_p = 2$					
1	0.31	0.82									
2	0.31	0.72	1.14								
3	0.31	0.65	0.99	1.33							
4	0.31	0.60	0.89	1.18	1.47						
5	0.31	0.56	0.82	1.07	1.33	1.59					
6	0.31	0.54	0.77	1.00	1.23	1.46	1.69				
7	0.31	0.50	0.70	0.89	1.09	1.28	1.48	1.67			
8	0.31	0.47	0.64	0.81	0.97	1.14	1.31	1.48	1.64		
9	0.31	0.45	0.59	0.73	0.87	1.01	1.16	1.30	1.44	1.58	
10	0.31	0.43	0.55	0.67	0.79	0.91	1.04	1.16	1.28	1.40	1.52
n						$\mu_p = 3$					
1	0.22	0.59									
2	0.22	0.52	0.81								
3	0.22	0.46	0.71	0.95							
4	0.22	0.43	0.63	0.84	1.05						
5	0.22	0.40	0.58	0.77	0.95	1.13					
6	0.22	0.38	0.55	0.71	0.88	1.04	1.21				
7	0.22	0.36	0.50	0.64	0.78	0.92	1.06	1.20			
8	0.22	0.34	0.46	0.58	0.70	0.82	0.94	1.05	1.17		
9	0.22	0.32	0.42	0.52	0.62	0.72	0.83	0.93	1.03	1.13	
10	0.22	0.31	0.39	0.48	0.57	0.65	0.74	0.83	0.91	1.00	1.09

Table C.5 (continued)

i	0	1	2	3	4	5	6	7	8	9	10
n	$\mu_p = 4$										
1	0.17	0.45									
2	0.17	0.40	0.63								
3	0.17	0.36	0.54	0.73							
4	0.17	0.33	0.49	0.65	0.81						
5	0.17	0.31	0.45	0.59	0.73	0.87					
6	0.17	0.30	0.42	0.55	0.68	0.80	0.93				
7	0.17	0.28	0.38	0.49	0.60	0.71	0.81	0.92			
8	0.17	0.26	0.35	0.45	0.54	0.63	0.72	0.81	0.91		
9	0.17	0.25	0.32	0.40	0.48	0.56	0.64	0.71	0.79	0.87	
10	0.17	0.24	0.30	0.37	0.44	0.50	0.57	0.64	0.70	0.77	0.84
n	$\mu_p = 5$										
1	0.14	0.37									
2	0.14	0.32	0.51								
3	0.14	0.27	0.44	0.60							
4	0.14	0.27	0.40	0.53	0.66						
5	0.14	0.25	0.37	0.48	0.60	0.71					
6	0.14	0.24	0.34	0.45	0.55	0.65	0.76				
7	0.14	0.23	0.31	0.40	0.49	0.58	0.66	0.75			
8	0.14	0.21	0.29	0.36	0.44	0.51	0.59	0.66	0.74		
9	0.14	0.20	0.26	0.33	0.39	0.46	0.52	0.58	0.65	0.71	
10	0.14	0.19	0.25	0.30	0.36	0.41	0.47	0.52	0.57	0.63	0.68
n	$\mu_p = 6$										
1	0.12	0.32									
2	0.12	0.28	0.44								
3	0.12	0.25	0.38	0.52							
4	0.12	0.23	0.34	0.46	0.57						
5	0.12	0.22	0.32	0.42	0.52	0.62					
6	0.12	0.21	0.30	0.39	0.48	0.57	0.65				
7	0.12	0.19	0.27	0.35	0.42	0.50	0.57	0.65			
8	0.12	0.18	0.25	0.31	0.38	0.44	0.51	0.57	0.64		
9	0.12	0.17	0.23	0.28	0.34	0.39	0.45	0.50	0.56	0.61	
10	0.12	0.17	0.21	0.26	0.31	0.35	0.40	0.45	0.50	0.54	0.59

APPENDIX D. Tables Related to NEHRP 1994

Table D.1 **Seismic Coefficient C_a**
(NEHRP Table 1.4.2.4a)

Soil Profile Type	$A_a<0.05$	$A_a=0.05$	$A_a=0.10$	$A_a=0.20$	$A_a=0.30$	$A_a=0.40$
A	A_a	0.04	0.08	0.16	0.24	0.32
B	A_a	0.05	0.10	0.20	0.30	0.40
C	A_a	0.06	0.12	0.24	0.33	0.40
D	A_a	0.08	0.16	0.28	0.36	0.44
E	A_a	0.13	0.25	0.34	0.36	0.36

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determined the value of C_a.

Table D.2 **Seismic Coefficient C_v**
(NEHRP Table 1.4.2.4b)

Soil Profile Type	$A_v<0.05$	$A_v=0.05$	$A_v=0.10$	$A_v=0.20$	$A_v=0.30$	$A_v=0.40$
A	A_v	0.04	0.08	0.16	0.24	0.32
B	A_v	0.05	0.10	0.20	0.30	0.40
C	A_v	0.09	0.17	0.32	0.45	0.56
D	A_v	0.12	0.24	0.40	0.54	0.64
E	A_v	0.18	0.35	0.64	0.84	0.96

NOTE: For intermediate values, the higher value or straight-line interpolation shall be used to determined the value of C_v.

Table D.3 **Architectural Component Coefficients**
(NEHRP Table 3.2.2)

Architectural Component or Element	a_p[a]	R_p[b]
1. Interior Nonbearing Walls and Partitions		
Stair and elevator enclosures	1	3.0
Other vertical enclosures	1	3.0
Area separation walls or fire walls	1	3.0
Plain (unreinforced) masonry walls	1	1.5
All other walls and partitions	1	3.0
2. Cantilever Elements		
Parapets	2.5	1.5
Chimneys	2.5	1.5
Stacks	2.5	3.0
3. Exterior Nonbearing Walls	1[d]	3.0
4. Exterior Wall Panels		
Panel	1	3.0
Connecting members and fasteners	1[d]	3.0
5. Veneer		
Ductile materials and attachments	1	4.0
Nonductile materials and attachments	1	1.5
6. Penthouses	2.5	4.0
7. Ceilings		
All	1	1.5
8. Racks and Cabinets		
Storage racks more than 8 ft (2.4 m) in height	2.5	4.0[c]
Storage racks detailed in accordance with the provisions of Chap. 5	2.5	6.0[c]
Storage cabinets and laboratory equipment	1	3.0
9. Access Floors		
Special access floors (designed in accordance with 1994 NEHRP section 3.2.7.2)	2.5	6.0
All other	2.5	3.0
10. Appendages and Ornamentations	1	3.0
11. Other Rigid Components		
Ductile materials and attachments	1	4.0
Nonductile materials and attachments	1	1.5
12. Other Flexible Components		
Ductile materials and attachments	2.5	4.0
Nonductile materials and attachments	2.5	1.5

[a] A lower value may be justified by detailed dynamic analysis. a_p shall not be less than 1.0.

[b] $R_p = 1.5$ for anchorage design when component anchorage is provided by expansion anchor bolts, shallow chemical anchors, or shallow (nonductile) cast-in-place anchors or when the component is constructed of nonductile materials. Powder-actuated fasteners (shot pins) shall not be used for component anchorage in Seismic Performance Categories D and E. Shallow anchors are those with an embedment length-to-bolt diameter ratio of less than 8.

[c] Storage racks over 8 ft in height shall be designed in accordance with the provisions of NEHRP section 3.2.9.1

[d] Where flexible diaphragms provide lateral support for walls and partitions, the value of a_p shall be increased to 2.0 for the center one-half of the span.

Table D.4 **Mechanical and Electrical Components Coefficients**
(NEHRP Table 3.3.2)

Mechanical and Electrical Component or Element	a_p[a]	R_p[b]
1. General Mechanical Equipment		
Boilers and furnaces	1	3.0
Pressure vessels on skirts and free-standing	2.5	3.0
Stacks	2.5	3.0
Cantilevered chimneys	2.5	1.5
Other	1	3.0
2. Manufacturing and Process Machinery		
General	1	3.0
Conveyors (nonpersonnel)	2.5	3.0
3. Piping Systems	2.5	4.0
4. Storage Tanks and Spheres		
Flat bottom (anchored)	2.5	4.0
Flat bottom (unanchored)	2.5	3.0
On braced or unbraced legs	2.5	2.0
5. HVAC System Equipment		
Vibration isolated	2.5	3.0
Nonvibration isolated	1	3.0
Mounted in-line with ductwork	1	3.0
Other	1	3.0
6. Elevator Components	1	3.0
7. Trussed Towers (free-standing or guyed)	2.5	3.0
8. General Electrical Equipment		
Communication	1	3.0
Bus ducts, conduit, cable tray	2.5	6.0
Panelboards, battery racks	2.5	3.0
Motor control centers, switchgear	2.5	3.0
Other	1	3.0
9. Lighting Fixtures	1	1.5

[a] A lower value may be justified by detailed dynamic analysis. a_p shall not be less than 1.0.

[b] R_p = 1.5 for anchorage design when component anchorage is provided by expansion anchor bolts, shallow chemical anchors, or shallow (nonductile) cast-in-place anchors or when the component is constructed of nonductile materials. Powder-actuated fasteners (shot pins) shall not be used for component anchorage in Seismic Performance Categories D and E. Shallow anchors are those with an embedment length-to-bolt diameter ratio of less than 8.

Table D.5 **Deflection Amplification Factor C_d and Response Modification Coefficient R**

Basic Structural System and Seismic Force Resisting Systems	**Response Mod. Coeff. R**	**Deflection Ampl. Factor C_d**
Bearing Wall System		
Light frame wall with shear panels	6.5	4
Reinforced concrete shear walls	4.5	4
Reinforced masonry shear walls	3.5	3
Concentrically braced frames	4	3.5
Plain (unreinforced) masonry shear walls	1.25	1.25
Plain concrete shear walls	1.5	1.5
Building Frame System		
Eccentrically braced frames, moment resisting connections at columns away from link	8	4
Eccen. braced frames, non-moment resisting connections at columns away from link	7	4
Composite eccentrically braced frames (C-EBF)	8	4
Light frame walls with shear panels	7	4.5
Concentrically braced frames	5	4.5
Composite concentrically braced frames (C-CBF)	5	4.5
Special concentrically braced frame of steel	6	5
Reinforced concrete shear walls	5.5	5
RC shear walls composite with steel elements	5.5	5
Steel plate reinforced composite shear walls	6.5	5.5
Reinforced masonry shear walls	4.5	4
Plain (unreinforced) masonry shear walls	1.5	1.5
Plain concrete shear walls	2	2
Moment Resisting Frame System		
Special moment frames of steel	8	5.5
Special moment frames of reinforced concrete	8	5.5
Special moment frame of masonry	5.5	5
Composite special moment frame (C-SMF)	8	5.5
Intermediate moment frames of reinforced concrete	5	4.5
Ordinary moment frames of steel	4.5	4
Composite ordinary moment frame (C-OMF)	4.5	4
Composite partially restrained frames (C-PRF)	6	5.5
Ordinary moment frames of reinforced concrete	3	2.5
Dual System w/ an SMF Capable of Resisting at least 25% of Prescribed Seismic Forces		
Eccentrically braced frames, moment resisting connections at columns away from link	8	4
Eccent. braced frames, non-moment resisting connections at columns away from link	7	4
composite eccentrically braced frames (C-EBF)	8	4
Concentrically braced frames	6	5
Special concentrically braced frames of steel	8	6.5
Composite concentrically braced frames (C-CBF)	6	5
Reinforced concrete shear walls	8	6.5
RC shear walls composite with steel elements	8	6.5
Steel plate reinforced concrete shear walls	8	6.5
Reinforced masonry shear walls	6.5	5.5
Wood sheathed shear panels	8	5
Dual System w/ an Intermediate RMF or a steel OMF Capable of Resisting at Least 25% of Prescribed Seismic Forces		
Concentrically braced frames	5	4.5
Composite concentrically braced frames (C-CBF)	5	4.5
Special concentrically braced frame of steel	6	5
Reinforced concrete shear walls	6	5
RC shear walls composite with steel elements	6	5
Steel plate reinforced composite shear walls	7	5.5
Reinforced masonry shear walls	5	4.5
Wood sheathed shear panels	7	4.5
Inverted Pendulum Structures-Seismic Force Resisting System		
Special moment frames of structural steel	2.5	2.5
Special moment frames of reinforced concrete	2.5	2.5
Ordinary moment frames of structural steel	1.25	1.25

Table D.6 **Allowable Story Drift, Δ_a (in. or mm)**

Building	**Seismic Hazard Exposure Group**		
	I	**II**	**III**
Buildings, other than masonry shear wall or masonry wall frame buildings, four stories or less in height with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall buildings	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall buildings	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
Masonry wall frame buildings	$0.013h_{sx}$	$0.013h_{sx}$	$0.0130h_{sx}$
All other buildings	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

APPENDIX E. Tables Related to Japan Code

Zone	A	B	C
C_Z	1.00	0.85	0.70

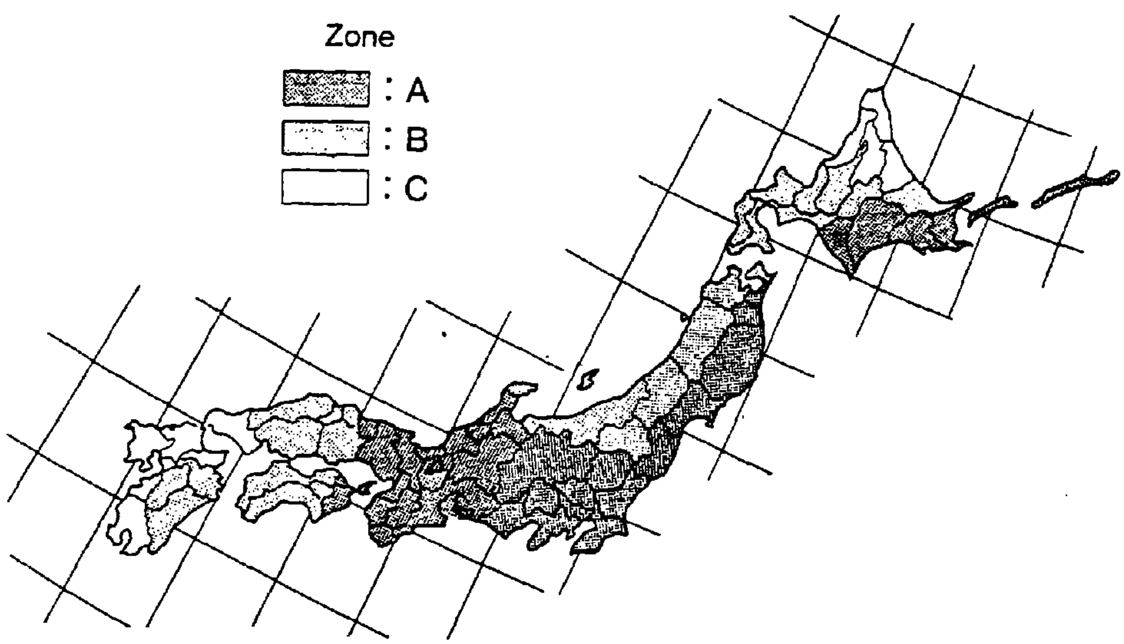

Figure E.1 **Seismic Zone Factor for Japanese Codes**

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