Chapter 13

Design of Nonstructural Systems and Components

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Abstract: For the majority of buildings, the nonstructural components represent a high percentage of the total capital investment. Failure of these components in an earthquake can disrupt the function of a building as surely as structural damage, and can pose a significant safety risk to building occupants as well. Past earthquakes have dramatically illustrated the vulnerabilities of the nonstructural components. Apart from the falling hazard posed by the light fixtures, non-structural failures can create debris that can block egress from the building, and hamper rescue efforts. In this Chapter, we deal chiefly with those components and systems that are installed in the structure during construction or remodel, for which design details are provided on the construction documents. We will touch briefly on the contents and equipment items that the owner or occupants may place in the building. The failure of these items may pose a significant risk to the occupants of the structure. However, these items are diverse, and the designer should address their anchorage and bracing on a case-by-case basis. Nonstructural elements can generally be divided into architectural, mechanical, and electrical systems and components. Architectural components include items such as exterior curtain walls and cladding, non-load bearing partitions, ceiling systems, and ornaments such as marquees and signs. Mechanical components and systems include boilers, fans, air conditioning equipment, elevators and escalators, tanks and pumps, as well as distributed systems such as HVAC (Heating, Ventilation, and Air Conditioning) ductwork and piping systems. Electrical components include transformers, panels, switchgear, conduit, and cable tray systems. Components may be mounted at grade (on the ground floor or basement of a building) or installed on the upper levels or roof of the structure. Our focus is on "nonstructural components" as opposed to "nonbuilding structures". Nonstructural components consist of equipment and systems that are supported vertically and laterally by a structural framework independent of the component itself -- a piece of equipment supported by a building frame, for example. In addition, we will consider the anchorage and bracing of moderately sized components at or below grade, such as chillers, pumps, and fans.

13.1 INTRODUCTION

For the majority of buildings, the nonstructural components represent a high percentage of the total capital investment. Failure of these components in an earthquake can disrupt the function of a building as surely as structural damage, and can pose a significant safety risk to building occupants as well. Past earthquakes have dramatically illustrated the vulnerabilities of the nonstructural components. Figure 13-1 illustrates the collapse of a suspending ceiling system in the 1971 San Fernando Earthquake. Apart from the falling hazard posed by the light fixtures, failures of this nature create debris that can block egress from the building, and hamper rescue efforts. Figure 13-2 shows a heavy rooftop tank that fell from its saddle mounts in 1994 Northridge Earthquake. Failure of this tank flooded the lower levels of the building.

In this Chapter, we deal chiefly with those components and systems that are installed in the structure during construction or remodel, for which design details are provided on the construction documents. We will touch briefly on the contents and equipment items that the owner or occupants may place in the building. The failure of these items may pose a significant risk to the occupants of the structure, as illustrated in Figure 13-3. However, these items are diverse, and the designer should address their anchorage and bracing on a case-by-case basis.

Nonstructural elements can generally be divided into architectural, mechanical, and electrical systems and components. Architectural components include items such as exterior curtain walls and cladding, non-load bearing partitions, ceiling systems, and ornaments such as marquees and signs. Mechanical components and systems include

Figure 13-1. Damaged suspended ceiling and light fixtures, Olive View Hospital, San Fernando Valley Earthquake of 1971 (Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

Figure 13-2. Rooftop tank failure, 1994 Northridge Earthquake

boilers, fans, air conditioning equipment, elevators and escalators, tanks and pumps, as well as distributed systems such as HVAC (Heating, Ventilation, and Air Conditioning) ductwork and piping systems. Electrical components include transformers, panels, switchgear, conduit, and cable tray systems. Components may be mounted at grade (on the ground floor or basement of a building) or installed on the upper levels or roof of the structure.

Our focus is on "nonstructural components" as opposed to "nonbuilding structures". Nonstructural components consist of equipment and systems that are supported vertically and laterally by a structural framework independent of the component itself -- a piece of equipment supported by a building frame, for example. In addition, we will consider the anchorage and bracing of moderately sized components at or below grade, such as chillers, pumps, and fans.

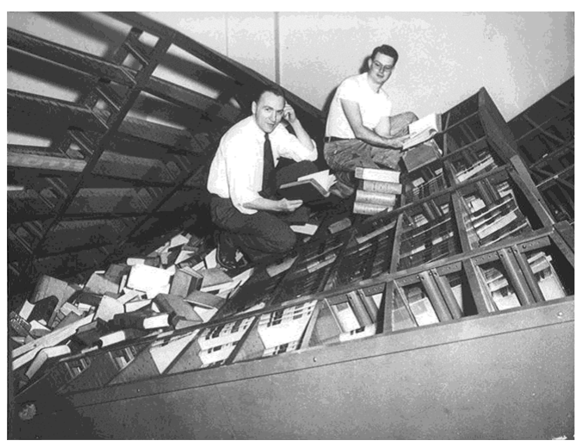


Figure 13-3. Overturned library shelves

Nonbuilding structures are supported on or below grade, and do not rely on another structure for vertical and lateral stability. Examples of nonbuilding structures include large industrial boilers and machinery, cooling towers, industrial storage rack systems, pressure vessels, and tanks. There are wide variations in the construction and dynamic properties of nonbuilding structures. Components such as pressure vessels, boilers, and chillers may be rigid structures, massively constructed with little inherent ductility. Seismic response of these components is often characterized by sliding or overturning at the level of connection to the ground. When damage occurs to these components, it is often concentrated in the connections or anchor bolts. At the opposite end of the spectrum are structures such as cooling towers, which are often flexible and highly redundant, with behavior quite similar to that for buildings.

The development of seismic design provisions for nonstructural components has lagged behind that of primary structural system. Until the advent of seismic codes, there was no clear distinction between structural and nonstructural components. Buildings had no dedicated lateral force resisting system, relying on plaster or brick walls and partitions for lateral strength. Earthquakes in the early part of the 20th century demonstrated the vulnerability of architectural features such as unreinforced brick parapets and exterior walls. Few observations were made regarding the seismic performance of mechanical and electrical systems, which existed in rudimentary forms.

In the 1933 Long Beach Earthquake, failure of fire sprinkler piping led to some of the earliest seismic provisions for piping systems. Lateral bracing provisions were added to the 1961 Uniform Building Code, dealing chiefly with the design and attachment of architectural components. However, it was not until the 1964 Alaska and 1971 San Fernando Earthquakes that the vulnerabilities of nonstructural components and systems in modern buildings were exposed. Earthquake reconnaissance reports from these and subsequent earthquakes identified many conditions and practices that caused extensive property damage and put building occupants at risk during strong ground shaking. Building code provisions have undergone continual development, incorporating lessons learned in these earthquakes. For example, the 1964 Alaska Earthquake demonstrated the vulnerabilities of precast concrete cladding systems. There were widespread failures of ceiling systems and mechanical equipment in the 1971 San Fernando Earthquake, and failures in piping systems in the 1994 Northridge Earthquake. After each of these events, building codes have been modified in an effort to address these vulnerabilities.

Structures that must continue in uninterrupted operation during and after an earthquake will require nonstructural component designs that exceed the levels in most building codes. In general, building codes treat equipment and systems as “black boxes”, in that while the seismic design for the item is limited to anchorage and bracing, the integrity of the component itself is not expressly considered. For example, seismic design of an electrical transformer typically consists of design of the anchor bolts connecting the unit to the structure, and perhaps a check of the mounting brackets on the transformer enclosure. However, checks of the integrity of the internal components of the unit are much less common, and are not required by building codes, even though the internal components may be acceleration sensitive and vulnerable to damage at acceleration levels significantly lower than the design anchorage force. For piping systems, bracing designed to prevent a collapse of the piping system may not be sufficient to prevent leaks or occasional breaks. The next generation of building codes will apply performance-based design to the anchorage and bracing of nonstructural components. In performance based-design, the design of a components or system is controlled by the level of seismic performance desired by the owner of the structure, or mandated by the governing building official.

Section 13.2 discusses performance objectives for different nonstructural components and systems. Section 13.3 examines different aspects of the seismic behavior of nonstructural components. Section

13.4 reviews the analytical approaches in different design standards. Sections 13-5 and 13-6 discuss some of the design characteristics of architectural and mechanical components and systems that have performed well in past earthquakes.

13.2 PERFORMANCE OBJECTIVES

The basic objective of seismic design is to provide an adequate level of safety, supplying protection that is appropriate for the seismic hazard and the importance of the component or system. Beyond this basic level of safety, which protects occupants from life threatening injury or death, higher levels of performance may be demanded, to limit damage or protect against loss of function. Tables 13-1 and 13-2 from FEMA 274 provide, for a range of architectural, mechanical, electrical, and plumbing systems and components, descriptions of damage states at different performance objectives. These descriptions depict the condition of the component or system following a design level earthquake.

For new construction, the minimum design objective should be Life Safety. Nonstructural components and systems in buildings constructed to this performance objective do not pose a significant threat to life, although the building may close for repairs following a strong earthquake. The emphasis is on elimination of falling hazards, but the nonstructural elements may not be functional or repairable following a strong earthquake.

Essential facilities, such as hospitals, police and fire stations, and emergency command centers may be designed with the intent that they meet the Immediate Occupancy or Operational performance objectives. Structures designed to these performance objectives are expected to be functional during or shortly after an earthquake. Interruption of lifeline services (public utilities such as electricity, water, and sewer) may disrupt the function of buildings designed to the Immediate Occupancy objective. Structures designed to the Operational performance objective generally have independent or back-up lifeline systems, and are not dependent on public utilities. When rehabilitating an existing structure, financial or physical constraints may limit the designer to the Hazards Reduced performance level, an objective somewhat below Life Safety.

Acceptance criteria for nonstructural components depend on the consequences of failure, and the performance level desired. For example, a water piping system may meet acceptance criteria for Life Safety if anchorage failures do not result in collapse of the system. The same installation will not meet the criteria for Immediate Occupancy, if the piping system develops leaks that will render the system inoperable. Components or systems containing significant amounts of hazardous materials require special care, since a breach may have catastrophic consequences.

Current building codes approach performance objectives indirectly. Buildings constructed to the minimum code provisions are expected to meet the Life Safety objective. Essential facilities are designed to more stringent standards. Component anchorage is designed for higher force levels, and a broader range of components may be subject to anchorage and bracing requirements. However, the desired performance objectives for essential facilities are sometimes unclear, and the relationship between the code provisions and performance objectives may not be defined.

Seismic design of nonstructural components is a balance between the potential losses versus the cost of damage mitigation measures. There are many cases where significant damage can be prevented by simply anchoring components to the floor or walls, at little cost. However, limiting damage to low levels in some components can be extremely costly. With the exception of essential facilities, economics should drive the selection of the design performance objective. An economic analysis should consider not only the direct cost of earthquake damage, but also indirect losses such as business interruption.

Table 13-1. Nonstructural Performance Levels and Damage – Architectural Components[13-2]

Component	Nonstructural Performance Levels			
	Hazards Reduced Level	Life Safety	Immediate Occupancy	Operational
Cladding	Severe damage to connections and cladding. Many panels loosened.	Severe distortion in connections. Distributed cracking, bending, crushing, and spalling of cladding elements. Some fracturing of cladding, but panels do not fall.	Connections yield; minor cracks (< 1/16" width) or bending in cladding.	Connections yield; minor cracks (< 1/16" width) or bending in cladding.
Glazing	General shattered glass and distorted frames. Widespread falling hazards.	Extensive cracked glass; little broken glass.	Some cracked panes; none broken.	Some cracked panes; none broken.
Partitions	Severe racking and damage in many cases.	Distributed damage; some severe cracking, crushing, and racking in some areas.	Cracking to about 1/16" width at openings. Minor crushing and cracking at corners.	Cracking to about 1/16" width at openings. Minor crushing and cracking at corners.
Ceilings	Most ceilings damaged. Light suspended ceilings dropped. Severe cracking in hard ceilings.	Extensive damage. Dropped suspended ceiling tiles. Moderate cracking in hard ceilings.	Minor damage. Some suspended ceiling tiles distrupted. A few panels dropped. Minor cracking in hard ceilings.	Generally negligible damage. Isolated suspended panel dislocations, or cracks in hard ceilings.
Parapets and Ornamentation	Extensive damage; some fall in nonoccupied areas.	Extensive damage; some fall in nonoccupied areas.	Minor damage.	Minor damage.
Canopies & Marquees	Extensive distortion.	Moderate distortion.	Minor damage.	Minor damage.
Chimneys & Stacks	Extensive damage. No collapse.	Extensive damage. No collapse.	Minor cracking.	Negligible damage.
Stairs & Fire Escapes	Extensive racking. Loss of use.	Some racking and cracking of slabs, usable.	Minor damage.	Negligible damage
Light Fixtures	Extensive damage. Falling hazards occur.	Many broken light fixtures. Falling hazards generally avoided in heavier fixtures (> 20 pounds)	Minor damage. Some pendant lights broken.	Negligible damage
Doors	Distributed damage. Many racked and jammed doors.	Distributed damage. Some racked and jammed doors.	Minor damage. Doors operable.	Minor damage. Doors operable.

Table 13-2. Nonstrucutral Performance Levels and Damage, Mechincal, Electrical, and Plumbing Systems/Components[13-2]

System/ Component	Nonstructural Performance Levels			
	Elevators out of service; counterweights off rails.	Elevators out of service; counterweights do no dislodge.	Elevators operable; can be started when power avilable.	Elevators operate.
HVAC Equipment	Most units do not operate; many slide or overturn; some suspended units fall.	Units shirt on supports, rupturing attached ducting, piping and conduit, but do not fall.	Units are secure and most operate if power and other required utilities are available.	Negligible damage.
Ducts	Ducts break loose of equipment and louvers; some supports fail; some ducts fall.	Minor damage at joints, with some leakage. Some supports damaged, but systems remain suspended	Minor damage at joints, but ducts remain serviceable.	Negligible damage.
Piping	Some lines rupture. Some supports fail. Some piping falls.	Minor damage at joints, with some leakage. Some supports damaged, but systems remain suspended.	Minor leaks develop at a few joints.	Negligible damage.
Fire Sprinkler Systems	Many sprinkler heads damaged by collapsing ceilings. Leaks develop at couplings. Some branch lines fail.	Some sprinkler heads damaged by swaying ceilings. Leaks develop at some couplings.	Minor leakage at a few heads or pipe joints. System remains operable.	Negligible damage.
Fire Alarm Systems	Ceiling mounted sensors damaged. System nonfunctional	May not function.	System is functional	System is functional
Emergency Lighting	Some lights fall. Power may not be available.	System is functional	System is functional	System is functional
Electrical Distribution Equipment	Units slide and/or overturn, rupturing attached conduit. UPS systems short out. Diesel generators do not start.	Units shift on supports and may not operate. Generators provided for emergency power start; utility service lost.	Units are secure and generally operable. Emergency generators start, but may not be adequate to service all power requirements.	Units are functional. Emergency power is provided, as needed.
Plumbing	Some fixtures broken; lines broken mains disrupted at source.	Some fixtures broken; lines broken mains disrupted at source.	Fixtures and lines serviceable; however, utility service may not be available.	System is functional. On-site water supply provided, if required.

13.3 NONSTRUCTURAL COMPONENT BEHAVIOR

Nonstructural components can be classified as deformation or acceleration sensitive. If the performance of a component is controlled by the supporting structure's deformation (typically measured by inter-story drift), it is deformation sensitive. Examples of deformation sensitive components include partitions, curtain walls, and piping systems running floor to floor. These components are often rigidly connected to the structure and span from floor to floor. Since they are vulnerable to racking and damage due to story drift, they are deformation sensitive.

When a component is not vulnerable to damage from inter-story displacements, such as a mechanical unit anchored to the floor of a structure, the component is acceleration sensitive. Acceleration sensitive components are vulnerable to shifting or overturning, if their anchorage or bracing is inadequate. The force provisions of building codes generally produce design forces high enough to prevent sliding, toppling, or collapse of acceleration sensitive components. Many components are both deformation and acceleration sensitive, although a primary mode of behavior can generally be identified. Table 13-3, taken from FEMA 274, identifies typical nonstructural components and whether they are acceleration or deformation sensitive.

Good seismic performance of deformation sensitive components can be obtained in two ways, by limiting the inter-story drift of the supporting structure, or by designing the component or system to accommodate the expected lateral displacements without damage. For higher structural performance objectives, the drift limit criteria for deformation sensitive components may govern the design of the primary lateral force-resisting system. In addition to considering the effects of lateral

Table 13-3. Nonstructural Components: Response Sensitivity [13-2]

	Component	Sensitivity Acc.	Sensitivity Def.
	A. Architectural		
1.	**Exterior Skin**		
	Adhered Veneer	S	P
	Anchored Veneer	S	P
	Glass Blocks	S	P
	Prefabricated Panels	S	P
	Glazing Systems	S	P
2.	**Partitions**		
	Heavy	S	P
	Light	S	P
3.	**Interior Veneers**		
	Stone, Including Marble	S	P
	Ceramic Tile	S	P
4.	**Ceilings**		
	a. Directly Applied to Structure	P	
	b. Dropped, Furred, Gypsum Board	P	
	c. Suspended Lath and Plaster	S	P
	d. Suspended Integrated Ceiling	S	P
5.	**Parapets and Appendages**	P	
6.	**Canopies and Marquees**	P	
7.	**Chimneys and Stacks**	P	
8.	**Stairs**	P	S

	Component	Sensitivity Acc.	Sensitivity Def.
	B. Mechanical Equipment		
1	**Mechanical Equipment**		
	Boilers and Furnaces	P	
	General Mfg. And Process Machinery	P	
	HVAC Equipment, Vibration Isolated	P	
	HVAC Equipment. Nonvibration Isolated	P	
	HVAC Equipment, Mounted In-line with Ductwork	P	
2.	**Storage Vessels and Water Heaters**		
	Structural Supported Vessels (Category 1)	P	
	Flat Bottom Vessels (Category 2)	P	
3.	**Pressure Piping**	P	S
4.	**Fire Suppression Piping**	P	S
5.	**Fluid Piping, not Fire Suppression**		
	Hazardous Materials	P	S
	Nonhazardous Materials	P	S
6.	**Ductwork**	P	S

Acc. = Acceleration-Sensitive
Def. = Deformation-Sensitive
P = Primary Response
S = Secondary Response

displacement of the primary structure, care must be taken that components and systems do not impact each other during the earthquake. Impact has been the source of widespread damage in past earthquakes. Interaction between components can be avoided by maintaining adequate clearances between flexibly supported equipment and systems. In addition, flexible couplings should be provided between rigid or braced components and those that are flexibly mounted or free to displace. Finally, displacement sensitive components and their connections must be designed to withstand their own inertial forces, generated by the earthquake.

Approximate median drift values that can be tolerated by different components are summarized in Table 13-4. The values given are median values based on recommendations in FEMA 273. These drift values are expected to generate severe damage to the nonstructural component at the Life Safety Performance Level, and moderate damage at the Immediate Occupancy Performance Level. The drifts are actual expected (unreduced) values. These deformations can be accommodated through flexible couplings, sliding joints, or through deformation of ductile elements in the component or system. The proximity of components to structural members and other systems must be considered. Distribution systems, such ducts, pipes, and conduits will "swing" between bracing points during ground shaking. Impacts between systems should be avoided, since they can cause support failures, and in piping systems, loss of contents.

Table 13-4. Drift Limits for Deformation Sensitive Components[13-1]

Component	Performance Objective	
	Life Safety	**Immediate Occupancy**
Adhered Veneer	0.03	0.01
Anchored Veneer	0.02	0.01
Nonstructural Masonry	0.02	0.01
Prefabricated Wall Panels	0.02	0.01
Glazing Systems	0.02	0.01
Heavy Partitions	0.01	0.005
Light Partitions	Not required	0.01
Interior Veneers	0.02	0.01

The amount of separation between components needed to prevent interaction should be determined. When determining the amount of separation needed, both the deformations of the bracing system and the deformations of the component between bracing points need to be considered. For example, the total displacement in a piping system should consider the deformation of the pipe braces and the deformations of the pipe itself between support points under the design seismic loading.

Design standards treat various types of nonstructural components individually, but interrelationships exist between components that must be considered. Failure of one component can precipitate failure of entire systems. For example, a pipe suffering a support failure may drop on to a suspended ceiling system, which in turn could fall into an exit corridor. Conversely, a well-braced pipe rigidly attached to a flexibly mounted pump can produce undesirable performance (Figure 13-4).

Acceleration sensitive components should be anchored or braced to the structure to prevent movement under the design loading. Care must be taken that these components are not anchored in such a way as to inadvertently affect the structural system. For example, if the base of a component with significant strength and stiffness is anchored to the floor and the top of the component is rigidly braced to the floor above, it can have the unintended effect of altering the response of the structural system. An example of this type of unintended interaction between a nonstructural component and the structural system is illustrated in Figure 13-5. The nonstructural masonry partition acts as a shear wall, which can lead to an unintended redistribution of lateral load. This condition could be avoided by providing isolation joints between the masonry wall and the structural columns wide enough to prevent interaction between the two elements, while providing a sliding connection at the top of the wall, which provides out-of-plane support but allows in-plane movement.

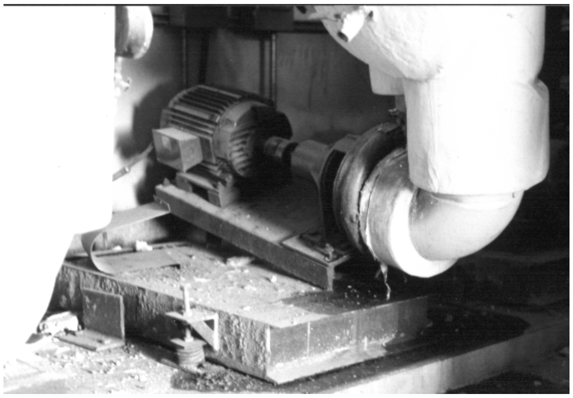


Figure 13-4. Failure of a flexibly mounted pump connected to a braced piping system, 1994 Northridge Earthquake

Components and systems mounted at or below grade respond to ground shaking in a fashion similar to buildings. The dynamic properties of the component (mass and stiffness) and the characteristics of the ground motion (frequency content, duration, etc.) govern their response. Behavior of components on the upper floors of buildings is complicated by the interaction of the dynamic characteristics of the structure and component. In cases where the mass of the nonstructural component is large in comparison with the overall mass of the structure, the techniques presented in this Chapter should be used with great caution. Large components may have a significant effect on the overall response of the structure. Depending on the dynamic properties of the component and the supporting structure, their dynamic response may be closely coupled. In general, if the component weight exceeds 20% of the total dead weight of the floor, or exceeds 10% of the total weight of the structure, the procedures discussed in this section should not be used. In such cases, the component and the

Figure 13-5. Nonstructural partition acting as a shear wall, 1964 Alaskan Earthquake. (Steinbrugge Collection, Earthquake Engineering Research Center, University of California, Berkeley.)

structure should be analyzed together, including proper representation of the flexibility of the component and it's supports.

Mechanical components with rotating or reciprocating components are often isolated from the structure with vibration isolation mounts. The isolation mounts may use either rubber-in-shear, springs, or air cushions to prevent transmission of vibrations to the structure. Vibration isolation mounts can dramatically alter the dynamic properties of components, by increasing their flexibility. Seismic inertial forces on isolated components are amplified. Improperly designed vibration isolation installations can fail under the increased dynamic and impact loads. Isolation mounts must be specifically designed to resist these effects. Housekeeping pads used to support equipment should be cast monolithically with the structural slab, or be adequately reinforced and doweled to the structural slab.

The ability to survive the earthquake physically intact does not guarantee the performance objective for the component or system has been met. As noted in Tables 13-1 and 13-2, a component or system may need to be functional following an earthquake in order to meet higher performance objectives. This requires seismic design of the operating parts of mechanical and electrical components, either through dynamic testing or through analysis. Systems relying on lifelines may require on-site back-up sources of water, emergency electrical power, and waste water storage to meet the Operational objective.

13.4 DESIGN STANDARDS

The development of analytical techniques for nonstructural components has mirrored that for the primary structure of buildings. Most of these techniques use equivalent lateral force methods, where the component is designed for a lateral seismic force that is expressed as a fraction of the component weight. Deformation sensitive components are designed to accommodate the design story drifts, amplified to the levels expected in the design earthquake. The objective of these approaches is to produce an anchorage or bracing scheme for the components that can withstand the accelerations generated by the earthquake, without allowing the component to shift or topple. In addition, the component must be able to tolerate the actual deformations of the primary structure without becoming dislodged, or adversely affecting the primary structure.

In this section we will examine the provisions of four design standards, the 1994 and 1997 editions of the *Uniform Building Code* (UBC), the *Tri-Services Manual*, and the 1997 *National Earthquake Hazards Reduction Program (NEHRP) Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, FEMA 302. These provisions provide the designer with guidance on typical nonstructural seismic issues, and may be used as resources on more complex or unusual projects.

Building codes may exempt components from anchorage and bracing requirements, depending on the level of seismic risk at the site, the occupancy of the structure, and the importance of the components. In regions of low seismicity, all components are typically exempt from seismic bracing requirements. In regions of moderate seismicity, bracing requirements are often limited to critical systems or hazardous components, such as cantilever parapets. In areas of high seismicity, furniture and components that are floor mounted and weigh less than 400 pounds are generally exempt from anchorage and bracing requirements. Items that or are suspended from the wall or ceiling and weigh less then 20 pounds are also typically exempt. However, exempt unanchored components may pose a risk, and consideration should be given to restraining items that could shift or topple, both for safety reasons and to limit property loss.

All components requiring anchorage should be designed for a minimum seismic force. Seismic forces are dependent on the following factors: component weight; flexibility or stiffness of the component and/or supports;

input acceleration at the point of attachment to the structure; an importance factor based on functionality requirements or the hazard posed by the item; and the ductility, redundancy and energy absorption capability of the component and its attachments to the structure. Positive restraints must be provided, and friction forces that are induced by gravity should be ignored, because vertical ground motions may reduce the effects of gravity. The effects of prying action and connection eccentricities on anchor loads should be accounted for in the design.

13.4.1 1994 UBC/Tri-Services Manual Static Analysis

Both the 1994 UBC and the Tri-Services Manual are based on procedures presented in the 1990 edition of the Structural Engineers Association of California Seismology Committee Recommendations. In the 1994 UBC, the design lateral force for components is given by the basic formula:

$$F_p = ZI_pC_pW_p \tag{13-1}$$

where:

F_p = lateral force applied to the center of mass of the component

Z = seismic coefficient that varies depending on the seismic zone in which the structure is located, and varies from 0.075 and 0.4

I_p = component importance factor, which depends on the occupancy of the structure and varies from 1.0 to 1.5

C_p = horizontal force factor, typically equal to 0.75 for most components, 2.0 for cantilever parapets and appendages,

W_p = weight of the component.

Components are classified as flexible or rigid, depending upon their dynamic characteristics. Rigid components are those with a fundamental period of vibration less than 0.06 seconds. Flexible components are those with higher fundamental periods. The values for C_p are amplified by a factor of 2 for flexible components, and may be reduced by 2/3 for components mounted at or below grade.

The response of components located on the upper levels is complicated by the dynamic response the structure in the ground shaking. Seismic input motion to the nonstructural component is filtered and amplified by the structure. This can produce dramatic amplifications of lateral force demands on the component, especially if the fundamental period of vibration of the component approaches a predominant mode of vibration of the supporting structure. In the Tri-Services Manual, an effort is made to more precisely consider the amplification of the seismic response experienced by flexible equipment on the upper levels of structures.

In the Tri-Services Manual, the force equation is modified to:

$$F_p = ZI_pA_pC_pW_p \tag{13-2}$$

where

A_p = magnification factor, dependent upon the ratio of the fundamental period of the component, T_a, and the period of the building T.

The component period may be determined by

$$T_a = 0.32\sqrt{\frac{W_p}{k}} \tag{13-3}$$

where

k = Stiffness of the equipment and/or the component supports, measured as kips per inch deflection of the center of gravity of the component, and

W_p = weight of the component, in kips.

The values of A_p vary from 1.0 to 5.0, depending on the relationship of the dynamic characteristics of the component and the supporting structure. If the dynamic properties of either the equipment or the structure are unknown, then a default value of A_p = 5.0 is used. For rigid components ($T_a \leq 0.06$ seconds), A_p = 1.0. When the period of non-rigid or flexibly mounted equipment is not known, but the fundamental period of the building is known, estimated values of A_p may be taken from Table 13-5, taken from Freeman, (1998).

Table 13-5. Estimated Amplification Factors, Ap Non-rigid and Flexibly Supported Equipment (Reference *13*-3)

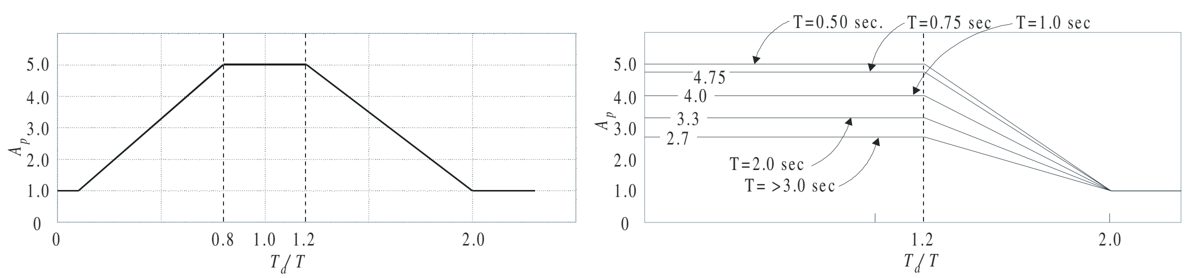
Builing Period T (seconds)	< 0.5	0.75	1.0	2.0	> 3.0
Ap	5.0	4.75	4	3.3	2.7

Where the dynamic properties of the structure and the equipment are known, then the value of A_p may be computed by first determining the fundamental period of the component, T_a using Equation 13-3. Then the ratio of T_a /T is determined, and the amplification factor A_p found from the appropriate curves from Figure 13-6, taken from the Tri-Services Manual.

Figure 13-6 shows the relationship between A_p and the ratio of the component to structure period. For a given component, the computation of the A_p factor can be somewhat involved, since higher modes of vibration of the structure must be considered. For structures with fundamental periods less than 2 seconds, Freeman recommends that A_p factors for the first, second, and third modes of vibration be computed. For structures with periods of greater than 2 seconds, the fourth and fifth modes should also be considered. The largest value of A_p governs. The product of $I_pA_pC_p$ need not exceed 3.75.

13.4.2 1997 UBC Analysis

The 1997 UBC provisions introduced significant changes in the design procedures for nonstructural components. These changes were driven by analysis of instrument records obtained from buildings that have experienced earthquake shaking. An examination of these records indicated that buildings experience a trapezoidal distribution of floor accelerations, varying linearly from the ground acceleration at the base to 3 or 4 times the ground acceleration at the roof. Figures 13-7 and 13-8, taken from FEMA 303, plot the amplification of peak acceleration versus height in the building based on data obtained from 405 building strong motion instrument records. Figure 13-7 shows the variation of the ratio of peak structural acceleration A to peak ground acceleration A_g versus height in the building for all records. Figure 13-8 shows the variation of the ratio of peak structural acceleration A to peak ground acceleration A_g versus height in the building for records where A_g exceeded 0.10g. The accelerations in both figures are mean values plus one standard deviation. The amplification of shaking as a function of height in the building is clearly shown. Other concepts introduced in the 1997 UBC include consideration of "near fault" and soils effects, use of Strength Design level loads, and introduction of an in-structure amplification factor, $a_{p,}$ which accounts for the force amplification effects experienced by flexible components.



(a) When the fundamental period of the building is equal or less than 0.5 seconds (T ≤ 0.5)

(b) When the fundamental period of the building is greater than 0.5 seconds (T > 0.5). (Note: If Ta / T < 1.2. Ap is equal to value obtained from Table 13.5

Figure 13-6. Amplification factor, Ap for nonrigid and flexibly supported equipment[13-5]

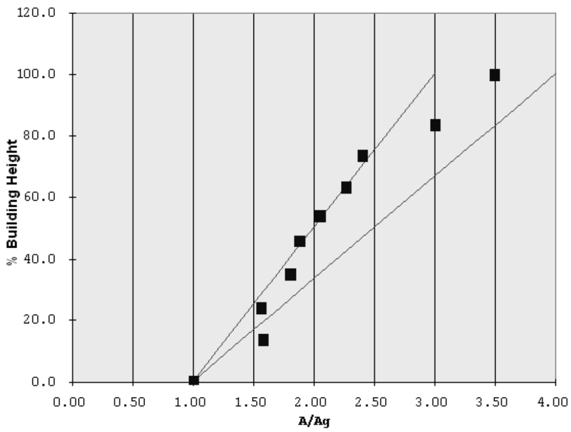


Figure 13-7. Amplification of peak ground acceleration (mean + 1σ) vs. building height(13-7)

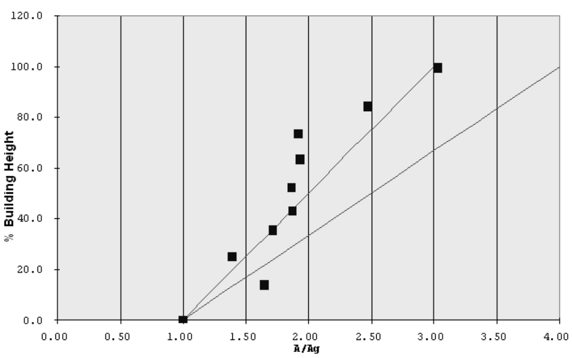


Figure 13-8. Amplification of peak ground acceleration (mean + 1σ) vs. building height, Ag > 0.10g(13-7)

The design lateral force for nonstructural components in the 1997 UBC is given by

$$F_p = \frac{a_p C_a I_p}{R_p}\left(1 + 3\frac{h_x}{h_r}\right)W_p \qquad (13\text{-}4)$$

where

F_p = lateral force applied to the center of mass of the component

a_p = in-structure amplification factor, that varies from 1.0 to 2.5

C_a = seismic coefficient that varies depending on the seismic zone in which the structure is located and the proximity to active earthquake faults. C_a varies from 0.075 to 0.66

I_p = component importance factor, which depends on the occupancy of the structure and varies from 1.0 to 1.5

R_p = component response modification factor, which varies from 1.5 to 3.0

h_x = element or attachment elevation with respect to grade, h_x shall not be taken as less than 0.

h_r = the structure roof elevation, with respect to grade

F_p shall not be less than $0.7C_aI_pW_p$, and need not exceed $4C_aI_pW_p$.

The a_p factor accounts for the dynamic amplification of force levels for flexible equipment. Rigid components, defined as components including attachments which have a period less than 0.06 seconds, are assigned an a_p = 1.0. Flexible components, which are defined as components including attachments which have a period greater than 0.06 seconds, are assigned an a_p = 2.5. Values of R_p are assigned based on the nature of the connections to the structure, as well as the properties of the component. Components fabricated of ductile materials and attachments may be assigned a R_p of 3. Components fabricated of nonductile materials or attachments are assigned a R_p of 1.5. Where connection of the component to concrete or masonry is made with shallow expansion, chemical, or cast-in-place anchors, R_p is taken as 1.5. Shallow anchors are defined as those anchors with an embedment length to diameter ratio of less than 8. If the anchors are constructed of brittle materials (such as ceramic elements in electrical components), or when anchorage is provided by adhesive, R_p is taken as 1.0. The term "adhesive" in this case refers to connections made using surface application of a bonding agent, and not anchor bolts embedded using epoxy or other adhesives. An example of anchorage made with adhesive would be post base plates glued to the surface of the structural floor in a raised access floor system. The design forces for equipment mounted on vibration isolation mounts must be computed using an a_p of 2.5 and a R_p of 1.5. If the isolation mount is attached to the structure using shallow or expansion-type anchors, the design forces for the anchors must be doubled.

In addition to lateral force requirements, the 1997 UBC specifies that for essential or hazardous facilities components must be designed for the effects of relative motion, if the component is attached to the structure at several points. An example would be a vertical riser in a piping system that runs from floor to floor. The component must accommodate the Maximum Inelastic Response Displacement, Δ_M, defined as:

$$\Delta_M = 0.7R\Delta_S \tag{13-5}$$

13.4.3 1997 NEHRP Analysis

The 1997 NEHRP provisions are similar in form to those in the 1997 UBC, although there are several significant differences. Many of these differences arise from the way ground shaking intensity is expressed. Rather than expressing ground shaking intensity through coefficients which are related to earthquake zones, the 1997 NEHRP expresses shaking intensity through peak spectral accelerations, which are mapped for long and short period structures. Because contour maps are used to present spectral accelerations, the increase in ground shaking intensity due to near-fault effects are directly accounted for, without the need for an additional factor in the force equation. As with the 1997 UBC, design loads are expressed at Strength Design (or Load and Resistance Factor Design) levels. Based on study of the records of instrumented buildings in areas of higher ground shaking intensity (Figure 13-8), the amplification of motion from the ground to roof levels was reduced from 4 to 3.

The design lateral force for nonstructural components in the 1997 NEHRP is given by:

$$F_p = \frac{0.4a_pS_{DS}W_p}{\dfrac{R_p}{I_p}}\left(1+2\frac{z}{h}\right) \tag{13-6}$$

where

F_p = lateral force applied to the center of mass of the component

a_p = in-structure amplification factor, that varies from 1.0 to 2.5

S_{DS} = spectral acceleration, short period

I_p = component importance factor, which depends on the component and occupancy of the structure and varies from 1.0 to 1.5

R_p = component response modification factor, which varies from 1.0 to 3.5.

z = element or attachment elevation with respect to grade. z shall not be taken as less than 0.

h = the average roof height of the structure relative to grade

F_p need not be greater than $1.6S_{DS}I_pW_p$, and may not be less than $0.3S_{DS}I_pW_p$. The a_p factor is defined in the same manner as that found in the 1997 UBC. Values of a_p and R_p for architectural and mechanical components are presented in Tables 13-6 and 13-7, respectively. When combining seismic and vertical loads, the reliability/redundancy factor, ρ, is taken as 1.0.

The component Importance Factor, I_p, is taken as 1.5 for life-safety components which must function after an earthquake, components with hazardous contents, storage racks in occupancies open to the public, and all components that could effect continued operation in essential (Seismic Use Group III) structures. For all other components, I_p, is taken as 1.0.

Values of R_p in the 1997 NEHRP are assigned based on the over-strength and deformability of the component's structure and attachments. Deformability is defined as the ratio of ultimate deformation to limit deformation. Ultimate deformation is the deformation at which failure occurs, and which is deemed to occur if the sustainable load reduces to 80 percent or less of the maximum strength. Limit deformation is defined as twice the initial deformation that occurs at a load equal to 40 percent of the maximum strength. Low deformability components have deformability of 1.5 or less, and are assigned a R_p = 1.25. High Deformability components

Table 13-6. Architectural Component Coefficients (FEMA 302)

Architectural Component or Element	a_p[a]	R_p[b]
Interior Nonstructural *Walls and Partitions*		
Plain (unreinforced) masonry *walls*	1.0	1.25
All other *walls and partitions*	1.0	2.5
Cantilever *Elements* (unbraced or braced to structural frames below its center of mass)		
Parapets and cantilever interior nonstructural *walls*	2.5	2.5
Chimneys and stacks where laterally supported by *structures*	2.5	2.5
Cantilever *Elements* (Braced to a structural frame above its center of mass		
Parapets	1.0	2.5
Chimneys and Stacks	1.0	2.5
Exterior nonstructural *walls*	1.0	2.5
Exerior Nonstructural *Wall Elements* and Connections		
Wall Element	1.0	2.5
Body of *wall* panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1
Veneer		
High deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.25
Penthouses (except when framed by an extension of the *building* frame)	2.5	3.5
Ceilings		
All	1.0	2.5
Cabinets		
Storage cabinets and laboratory equipment	1.0	2.5
Access floors		
Special access floors	1.0	2.5
All other	1.0	1.25
Appendages and Ornamentations	2.5	2.5
Signs and Billboards	2.5	2.5
Other Rigid *Components*		
High deformability elements and *attachments*	1.0	3.5
Limited deformability elements and *attachments*	1.0	2.5
Low deformability elements and *attachments*	1.0	1.25
Other *flexibile components*		
High deformability elements and *attachments*	2.5	3.5
Limited deformability elements and *attachments*	2.5	2.5
Low deformability elements and *attachments*	2.5	1.25

[a]A lower value for a_p may be justified by detailed dynamic analysis. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid and rigidly attached. The value of $a_p = 2.5$ is for *flexibile components* of flexibly attached *components* and *flexibile components* including *attachments*.

[b]$R_p = 1.25$ 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. Power-actuated fasteners (shot pins) shall not be used for *component* anchorage in tension applications in *Seismic Design Categories* D, E or F. *Shallow anchors* are those with an embedment length-to-diameter ratio of less than 8.

have deformability greater than 3.5 when subjected to four fully reversed cycles at the limit deformation, and are assigned an $R_p = 3.5$. Limited deformability components, defined as components that have neither high nor low deformability, are assigned a $R_p = 2.5$. The design force F_p for vibration isolated components must be doubled.

Component anchorage to concrete and masonry are subject to additional requirements. Anchors embedded in concrete or masonry must be proportioned to carry the least of the following:

– The design strength of the connected part, or
– Two times the force in the connected part due to the prescribed forces, or

Table 13-7. Mechanical and Electrical Component Coefficients (FEMA 302)

Mechanical and Electrical Component or Element[c]	a_p[a]	R_p[b]
General Mechanical		
Boilers and furnaces	1.0	2.5
Pressure vessels on skirts and free-standing	2.5	2.5
Stacks	2.5	2.5
Cantilevered chimneys	2.5	2.5
Other	1.0	2.5
Manufacturing and Process Machinery		
General	1.0	2.5
Conveyors (nonpersonnel)	2.5	2.5
Piping Systems		
High deformability elements and *attachments*	1.0	3.5
Limited deformability elements and *attachments*	1.0	2.5
Low deformability elements and *attachments*	1.0	1.25
HVAC System Equipment		
Vibration isolated	2.5	2.5
Nonvibration isolated	1.0	2.5
Mounted in-line with ductwork	1.0	2.5
Other	1.0	2.5
Elevator *Components*	1.0	2.5
Escalator *Components*	1.0	2.5
Trussed Towers (free-standing or guyed)	2.5	2.5
General Electrical		
Distributed systems (bus ducts, conduit, cable tray)	2.5	3.5
Equipment	1.0	2.5
Lighting Fixtures	1.0	1.25

[a]A lower value for a_p is permitted provided a detailed dynamic analysis is performance which justifies a lower limit. The value for a_p shall not be less than 1.00. The value of $a_p = 1$ is for equipment generally regarded as rigid or rigidly attached. The value of $a_p = 2.5$ is for *fexibile components* or flexibly attached *components*.

[b]R_p = 1.25 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. Power-actuated fasteners (shot pins) shall not be used for *component* anchorage in tension applications in *Seismic Design Categories* D, E or F. *Shallow anchors* are those with an embedment length-to-diameter ratio of less than 8.

[c]*Components* mounted on vibration isolation systems shall have a bumper restraint or snubber in each horizontal direction. The design force shall be taken as $2F_p$.

– The maximum force that can be transferred to the connected part by the component structural system.

Components must also meet requirements for relative displacements. Seismic relative displacement, D_p, is defined as

$$D_p = \delta_{xA} - \delta_{yA} \qquad (13\text{-}7)$$

Where

δ_{xA} = deflection at building level x of the structure, determined by elastic analysis and multiplied by the C_d factor

δ_{yA} = deflection at building level y of the structure, determined by elastic analysis and multiplied by the C_d factor

D_p need not exceed

$$D_p = (X - Y)\frac{\Delta_{aA}}{h_{sx}} \qquad (13\text{-}8)$$

Where

X = height of upper support attachment at level x as measured from the base.

Y = height of upper support attachment at level y as measured from the base.

h_{sx} = story height used in the definition of the allowable drift

Δ_{aA} = allowable story drift of for the structure

The provisions for cases where the connection points are on two separate structures are developed in a similar manner.

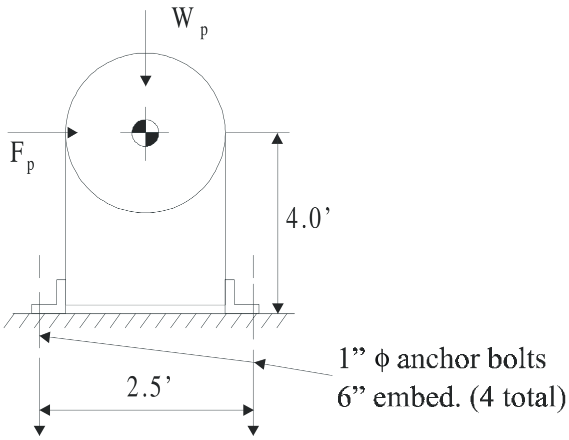


Figure 13-9. Boiler Example

Example 13-1

A steam boiler will be installed in the mechanical penthouse on the roof of a 4-story building. The dimensions of the unit are shown in Figure 13-9. The fundamental period of the boiler is 0.04 seconds. There are 4 one-inch diameter anchor bolts, one at each corner of the boiler, embedded in a concrete slab. The bolts have an embedment length of 6 inches. The building is in a region of high seismicity, UBC Seismic Zone 4. Per the 1997 UBC, the site is within 5 kilometers of a "Type B" seismic source, and located on Soil Profile Type S_D. Per the 1997 NEHRP, the 0.2 second spectral response acceleration is Ss = 175% g.

1. Using the 1994 UBC provisions, determine the shear and tension demands on the anchor bolts.

Z = 0.4 (Seismic Zone 4)
I_p = 1.0 (Standard occupancy structure)
C_p = 0.75
W_p = 20.0 kips

The period of the component is less than 0.06 seconds, so the equipment is considered rigid. The design lateral force for the component, determined by Equation 13-1 is

$$F_p = ZI_pC_pW_p$$
$$= 0.4(1.0)(0.75)(20.0 \text{ kips})$$
$$= 0.3(20.0 \text{ kips}) = 6.0 \text{ kips}$$

The shear per anchor bolt is

$V = F_p/4$ = (6.0 kips)/4 = 1.5 kips per anchor bolt.

The overturning moment is

M_{ot} = (6.0 kips)(4.0 ft) = 24.0 kip-ft

and the resisting moment is

M_r = (0.85)(20.0 kips)(1.25 ft) = 21.3 kip-ft.

Note that the resisting moment is reduced 15%, to take into account the effects of vertical acceleration. Taking the sum of the moments about a corner of the base, the uplift force F_t in the anchors equals

$$Ft = \frac{24.0 - 21.3}{(2.5)(2 \text{ anchor bolts / side})} = 0.54 \text{ kips}$$

Note that the Tri-Services Manual will produce identical results, since for this case A_p = 1.0.

2. Using the 1997 UBC provisions, determine the shear and tension demands on the anchor bolts, assuming the bolts will be designed using Allowable Stress procedures.

$h_x = h_r$ = 40 feet (roof top installation)
I_p = 1.0 (Standard occupancy structure)
a_p = 1.0 (rigid component)
W_p = 20.0 kips
l_e/d_b = 6.0/1.0 = 6 (the ratio of anchor bolt embedment length/bolt diameter)

For Soil Profile Type S_D, C_a = 0.44N_a, where N_a is the near source factor. Our site is within 5 kilometers of a "Type B" seismic source, so N_a = 1.0. Therefor C_a = 0.44. Rp=1.5, because the ratio of anchor bolt embedment depth to diameter of 6 is less than 8.

The design lateral force for the component determined by Equation 13-4 is

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

$$F_p = \frac{(1.0)(0.44)(1.0)}{1.5}\left(1+(3)\frac{40}{40}\right)W_p$$

$= 1.17(20.0 \text{ kips}) = 23.4 \text{ kips}$

The shear per anchor bolt, is

$V = F_p/4 = (23.4 \text{ kips})/4 = 5.9$ kips per anchor bolt.

The overturning moment is

$M_{ot} = (23.4 \text{ kips})(4.0 \text{ ft}) = 93.6$ kip-ft

and the resisting moment is

$M_r = (0.9)(20.0 \text{ kips})(1.25 \text{ ft}) = 22.5$ kip-ft.

Note that in this case, the resisting moment is reduced 10%, to take into account vertical accelerations. Taking the sum of the moments about a corner of the base, the uplift force F_t in the anchors equals

$$F_t = \frac{93.6 - 22.5}{(2.5)(2 \text{ anchors / side})} = 14.2 \text{ kips}$$

To convert these shear and tension forces to Allowable Stress Design levels, we divide by a factor of 1.4, to obtain

$V = 5.9 \text{ kips}/1.4 = 4.2$ kips

$F_t = 14.2 \text{ kips}/1.4 = 10.1$ kips

3. Using the 1997 NEHRP, determine the shear and tension demands on the anchor bolts, assuming the bolts will be designed using Allowable Stress procedures.

$z = h = 40$ feet (roof top installation)

$I_p = 1.0$ (Standard occupancy structure)

$a_p = 1.0$ (rigid component)

R_p=1.25, because the ratio of anchor bolt embedment depth to diameter is less than 8. For Soil Profile Type S_D, $S_{MS} = 1.0S_S$, and $S_{DS} = (2/3)S_{MS}$. Therefore,

$S_{DS} = (2/3)(1.75 \text{ g}) = 1.17$ g

The design lateral force for the component determined by Equation 13-6 is

$$F_p = \frac{0.4a_pS_{DS}W_p}{\dfrac{R_p}{I_p}}\left(1+2\frac{z}{h}\right)$$

$$F_p = \frac{(0.4)(1.0)(1.17)}{\dfrac{1.25}{1.0}}\left(1+(2)\frac{40}{40}\right)W_p$$

$= 1.12(20.0 \text{ kips}) = 22.5 \text{ kips}$

The shear per anchor bolt is

$V = F_p/4 = (22.5 \text{ kips})/4 = 5.6$ kips per anchor bolt.

The overturning moment is

$M_{ot} = (22.5 \text{ kips})(4.0 \text{ ft}) = 90.0$ kip-ft

and the tension per bolt from overturning is

$$Ft = \frac{90.0}{(2.5)(2 \text{ anchors / side})} = 18.0 \text{ kips}$$

and the dead load tributary to each anchor bolt is

$F_D = 20.0 \text{ kips}/4 \text{ bolts} = 5.0$ kips

The gravity load is reduced by $0.2S_{DS}D$ to account for the effects of vertical seismic accelerations. The net tension per bolt is

$T = 18.0 - [5.0-(0.2)(1.17)(5.0)] = 14.2$ kips per bolt.

To convert these shear and tension forces to Allowable Stress Design levels, we divide by a factor of 1.4, to obtain

$V = 5.6 \text{ kips}/1.4 = 4.0$ kips

$F_t = 14.2 \text{ kips}/1.4 = 10.1$ kips

The design shear and tension demands on the bolt must be doubled unless the design strength of the connected part or the maximum force that can be delivered by the component structural system limits the load to anchor bolts. An example of a mechanism that could limit the force to the bolt would be yielding of a steel base plate or bracket. For the purposes of comparison, we assume that the base plate yields at the design load.

The results obtained from the four methods are summarized in Table 13-8. Clearly, the design of rigid, acceleration sensitive components has become significantly more conservative in the 1997 UBC and NEHRP provisions. Design bolt shears in our example increase by 126% and 180% respectively, using the 1997 NEHRP and 1997 UBC. Increases in the design uplift demands on the anchor bolts increase even more dramatically, over 18 times the 1994 UBC provisions using the 1997 UBC.

A portion of these increases can be attributed to changes in the characterization of ground shaking in regions of high seismic risk. In addition, the 1997 UBC and NEHRP provisions include factors to account for amplification of ground motion in the upper portions of structures. Finally, the 1997 provisions attempt to refine and rationalize the reduction factors (R_p). Individually, each of these changes can be justified, but collectively, the produce very conservative results, that are difficult to justify in the light of experience in recent earthquakes.

Table 13-8. Summary, Example 1 Results

Method	Bolt Shear	Bolt Tension
1994 UBC	1.5 kips	0.54 kips
Tri-Services Manual	1.5 kips	0.54 kips
1997 UBC	4.2 kips	10.1 kips
1997 NEHRP	4.0 kips	10.1 kips

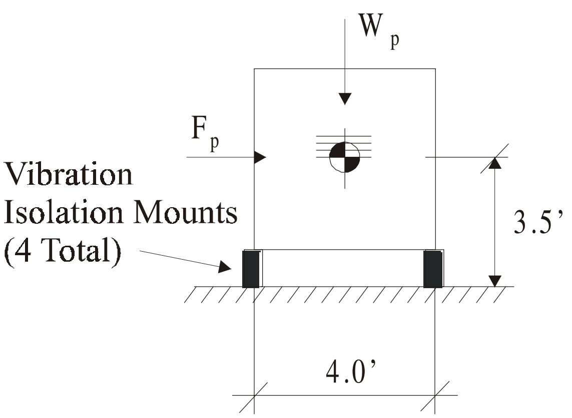


Figure 13-10. Emergency Generator Example

Example 13-2

An electrical generator is installed on the third floor of a 5-story emergency command center. The dimensions of the unit are shown in Figure 13-10. The generator is mounted on four vibration isolation mounts (one at each corner of the unit), with a lateral stiffness of 3 kips/inch each. The building floor-to-floor height is 12 feet, and the fundamental period of the building is 0.5 seconds. The building is in UBC Seismic Zone 3, a region of moderately high seismicity, and is not in the proximity of an active fault.. Per the 1997 NEHRP, the 0.2 second spectral response acceleration is Ss = 100% g. The site has been identified as Soil Profile Type S_D

1. Using the 1994 UBC provisions, determine the shear and tension demands on the vibration isolation mounts.

Z = 0.3 (Seismic Zone 3)
I_p = 1.5 (essential occupancy structure)
C_p = 0.75
W_p = 15.0 kips

The period of the equipment can be estimated using equation (*13*-3):

$$T_a = 0.32\sqrt{\frac{W_p}{k}}$$

$$T_a = 0.32\sqrt{\frac{15.0 \text{ kips}}{(3.0 \text{ kips / inch})(4 \text{ mounts})}}$$

$$T_a = 0.36 \text{ seconds}$$

The period of the component is greater than 0.06 seconds, so the equipment is considered flexible and the value of C_p must be multiplied by a factor of 2. The design lateral force for the component using Equation *13*-1 is

$$F_p = ZI_pC_pW_p$$
$$= 0.3(1.5)(2 \text{ x } 0.75)(15.0 \text{ kips})$$
$$= 10.2 \text{ kips}$$

The shear per vibration isolation mount is

$V = F_p/4$ = (10.2 kips)/4 = 2.6 kips per mount.

The overturning moment is

M_{ot} = (10.2 kips)(3.5 ft) = 35.7 kip-ft

and the resisting moment is

M_r = (0.85)(15.0 kips)(2.0 ft) = 25.5 kip-ft.

Taking the sum of the moments about a corner of the base, the uplift force F_t in the vibration isolation mount equals

$$Ft = \frac{35.7 - 25.5}{(4.0)(2 \text{ mounts/side})}$$
$$= 1.3 \text{ kips per mount}$$

2. Using the provisions of the Tri-Services Manual, determine the shear and tension demands on the vibration isolation mounts.

Since T_a = 0.36 seconds, the equipment is considered flexibly mounted. The ratio of component period to fundamental period of the structure is

$$\frac{T_a}{T} = \frac{0.36}{0.5} = 0.72$$

Entering the graph in Figure 13-6, we obtain a value for A_p = 4.5. It is unlikely that the higher modes of vibration of the building will produce a greater value of A_p. Then the design lateral force using Equation 13-2 is

$$F_p = ZI_p A_p C_p W_p$$

$$= (0.3)(1.5)(4.5)(0.75)\mathrm{W_p}$$

$$= 1.52\ \mathrm{W_p}$$

However, $I_pA_pC_p$ need not exceed 3.75, which in this example governs. Substituting these values into Equation 13-2, we find

$$F_p = (0.3)(3.75)\mathrm{W_p} = 1.13\ \mathrm{W_p} = 16.9 \text{ kips}$$

The shear per vibration isolation mount is

$V = F_p/4$ = (16.9 kips)/4 = 4.2 kips per mount.

The overturning moment is

M_{ot} = (16.9 kips)(3.5 ft) = 59.2 kip-ft

and the resisting moment is

M_r = (0.85)(15.0 kips)(2.0 ft) = 25.5 kip-ft.

Summing moments about a corner of the base, the uplift force F_t in the vibration isolation mount equals

$$Ft = \frac{59.2 - 25.5}{(4.0)(2 \text{ mounts/side})}$$

$$= 4.2 \text{ kips per mount}$$

3. Using the 1997 UBC provisions, determine the shear and tension demands on the vibration isolation mounts, assuming they will be designed using Allowable Stress procedures.

h_x = (3 floors)(12 feet/floor) =36 feet

h_r = (5 floors)(12 feet/floor) = 60 feet

I_p = 1.5 (essential occupancy structure)

a_p = 2.5 (flexible component)

R_p = 1.5 (vibration isolated component)

W_p = 15.0 kips

N_a = 1.0 (no nearby faults)

For Soil Profile Type S_D, $C_a = 0.36N_a = 0.36$. Substituting these variables into Equation (13-4), the design lateral force for the component using Equation 13-4 is

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

$$F_p = \frac{(2.5)(0.36)(1.5)}{1.5}\left(1 + (3)\frac{36}{60}\right)W_p$$

$$= 2.52\ W_p$$

However, F_p need not exceed $4C_aI_pW_p$, so

F_p= 4(0.36)(1.5) W_p = 2.16 W_p = 32.4 kips

The shear per isolation mount is

$V = F_p/4$ = (32.4 kips)/4 = 8.1 kips per isolation mount.

The overturning moment is

M_{ot} = (32.4 kips)(3.5 ft) = 113.4 kip-ft

and the resisting moment is

M_r = (0.9)(15.0 kips)(2.0 ft) = 27.0 kip-ft.

Summing the moments about a corner of the base, the uplift force F_t in the vibration isolation mount equals

$$Ft = \frac{113.4 - 27.0}{(4.0)(2 \text{ anchors / side})} = 10.8 \text{ kips}$$

To convert these shear and tension forces to Allowable Stress Design levels, we divide by a factor of 1.4, to obtain

V = 8.1 kips/1.4 = 5.8 kips

F_t = 10.8 kips/1.4 = 7.7 kips

4. Using the 1997 NEHRP/FEMA 273, determine the shear and tension demands on the vibration isolation mounts, assuming the mounts will be designed using Allowable Stress procedures.

z = 36 feet

h = 60 feet

I_p = 1.5 (essential component)

a_p = 2.5 (flexible component)

R_p = 2.5

W_p = 15.0 kips

For Soil Profile Type S_D, $S_{MS} = 1.1S_S$, and $S_{DS} = (2/3)S_{MS}$. Therefore,

S_{DS} = (2/3)(1.1)(1.00 g) = 0.73 g

The design lateral force for the component from Equation 13-6 is

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

$$F_p = \frac{(0.4)(2.5)(0.73)}{\frac{2.5}{1.5}}\left(1 + (2)\frac{36}{60}\right)W_p$$

= 0.96(15.0 kips) = 14.4 kips

Since the component is mounted on vibration isolators, the design force is doubled, so

F_p = (2)(14.4 kips) = 28.8 kips

The shear per isolation mount is

$V = F_p/4$ = (28.8 kips)/4 = 7.2 kips per isolation mount.

The overturning moment is

M_{ot} = (28.8 kips)(3.5 ft) = 100.8 kip-ft

and the tension per mount from overturning is

$$Ft = \frac{100.8}{(4.0\text{ feet})(2\text{ mounts/side})} = 12.8\text{ kips}$$

The dead load tributary to each isolation mount is

F_D = 15.0 kips/4 mounts = 3.8 kips

The gravity load is reduced by $0.2S_{DS}D$, and the net tension per isolation mount is

T = 12.6 - [3.8-(0.2)(0.73)(3.8)] = 9.4 kips per mount.

To convert these shear and tension forces to Allowable Stress Design levels, we divide by a factor of 1.4, to obtain

V = 7.2 kips/1.4 = 5.1 kips

F_t = 9.4 kips/1.4 = 6.6 kips

The results obtained from the four methods for this example are summarized in Table 13-9. Again, this example shows the four methods can produce results that differ significantly. The 1994 UBC is by far the simplest method, and yields the lowest design forces. The other three methods add significant complexity, and produce higher design forces. In this example, the higher design forces may be justified, since there have been a number of failures of vibration isolated equipment in recent earthquakes. As with Example 13-1, a portion of the increase in design force using the 1997 UBC and the 1997 NEHRP can be attributed to changes in the design ground shaking intensities. The amplification of design forces in the upper levels of the structure is in keeping with strong motion data obtained from recent earthquakes. Figure 13-8 shows that a linear amplification of ground acceleration by a factor of three from the ground to roof levels (as used in the 1997 NEHRP) bounds instrument records well, while the amplification factor of 4 used in the 1997 UBC is conservative. The 1994 UBC and Tri-Services approaches ignore this phenomenon.

Table 13-9. Summary, Example 2 Results

Method	Bolt Shear	Bolt Tension
1994 UBC	2.6 kips	1.3 kips
Tri-Services Manual	4.2 kips	4.2 kips
1997 UBC	5.8 kips	7.7 kips
1997 NEHRP	5.1 kips	6.6 kips

13.5 DESIGN CONSIDERATIONS FOR ARCHITECTURAL COMPONENTS

13.5.1 General

Architectural nonstructural components include items such as exterior curtain walls and cladding; non-load bearing partitions; ceiling systems; and ornaments such as marquees and signs. In addition, they can include a wide array of shelving, cabinets, workstations, and equipment that are installed by the building occupant.

For life safety, the objective of the design should be to limit the severity of damage to the architectural components so that they do not topple, or detach themselves from the structure and fall. For higher performance objectives, it may be necessary to control damage to the components so that functionality is not impaired. For example, a curtain wall system that does not fall from the building or block egress may be considered to have met a life safety performance objective. For immediate occupancy, it may be necessary to limit damage of the system so that it continues to be weather-tight.

Much of the information in the following sections has been adopted from the excellent discussion of nonstructural components found in FEMA 273 and FEMA 274.

Some architectural components are inherently vulnerable to earthquake damage. For example, cracking will occur in stucco and plaster at relatively low levels of ground shaking. Limiting building drift, or providing component anchorage to protect these materials from damage, is generally not cost-effective. Much damage can be minimized through careful detailing of the components. The objective is to minimize the amount of distortion experienced by the element due story drift, and for acceleration-sensitive items, provide adequate anchorage to prevent shifting or toppling. With proper attention to detailing, damage in moderate ground shaking can be limited to a level that is easily and inexpensively repaired.

Damage to building contents outside the scope of the designer, such as furniture, countertop items (for example, computers), cabinets, and shelving can be limited by providing adequate anchorage for these items. The contents of cabinets and shelving can be restrained. However, most items that are portable are difficult to anchor effectively for seismic forces. People using these items will often prefer not to employ the seismic latches, tethers, or other restraint devices provided, since they generally make the use of the item less convenient.

13.5.2 Architectural Finishes

Plaster and stucco are common finish materials that are very brittle. At relatively low displacements, plaster begins to crack. As displacements increase and the finish is further distorted, the material spalls, and can separate from the supporting lath. Plaster directly applied over structural elements that form part of the lateral force-resisting system is especially vulnerable. In a large earthquake, the structural elements are expected to experience inelastic behavior, and the distortions of the elements associated with this behavior will usually cause significant damage to the plaster finish. Generally, repairs to the plaster finishes are inexpensive, and the damage does not represent a significant hazard. However, failure of a large plaster or gypboard surface, such as a ceiling, can pose both a falling hazard and block the path of egress. Ceiling systems should be designed to accommodate the expected distortions of the supporting structure without collapse. Where significant diaphragm distortions are expected, consideration should be given to isolating the furring for plaster ceilings from the diaphragm.

Shear cracking of surface finishes near doors and windows is a common form of earthquake damage, and is probably unavoidable. Although this type of damage is most common in plaster surfaces, other wall finishing materials are vulnerable. Post-earthquake repairs are relatively inexpensive, provided matching materials are available. For tile finishes, finding a suitable matching tile for repairs can be difficult.

Adhered veneer refers to thin surface materials, such as tile, thin set brick, or stone, which rely on adhesive attachment to a backing or substrate for support. This includes tile, masonry, stone, terra cotta and similar materials not over 1 inch in thickness, as well as ceramic tile and exterior plaster (stucco). These materials are supported by adhesive (not mechanical) attachment to a supporting substrate, which may be masonry, concrete, cement plaster, or a structural framework. Adhered veneers are deformation sensitive, and their seismic performance depends on the performance of the supporting substrate. Adhered veneer materials are often inherently brittle. Deformation of the substrate leads to cracking, which can result in the veneer separating from the substrate. The key to good seismic performance is to detail the substrate so as to isolate it from the effects of story drift. The materials are most vulnerable at discontinuities, such as corners and openings.

The threat to life safety posed by adhered veneers depends on the height of the veneer, the size and weight of the fragments likely to become dislodged, and the nature of the occupancy. It is important to distinguish between falling of individual units such as tiles, which typically would not be considered a life-safety issue, and large areas of the veneer separating from the substrate and falling.

Anchored veneer consists of masonry units that are attached to the supporting structure by mechanical means. This type of veneer is both acceleration and deformation sensitive. The masonry units can be dislodged by accelerations which distorts or fail the mechanical connectors. Deformations of the supporting structure may displace or dislodge the units by racking. Damage to anchored veneers can be controlled by limiting the drift ratios of the supporting structure, isolating units from story drift through slip connections or joints, and by anchoring the veneers for an adequate force level that includes consideration of the vertical component of ground shaking. Special attention should be paid at locations likely to experience large deformations, especially at corners and around openings.

Masonry veneer facades on steel frame buildings should be avoided unless the veneer is securely tied to a separate wall or framework that is independent of the primary (gravity and lateral load carrying) steel frame. Otherwise, adequate provisions for the large expected lateral deformation of the steel frame must be made. Wire or straight rod ties should not be used to anchor face brick to a wall, especially when a layer of insulation or an air gap separates the two elements. Large masonry facades may be designed as part of the structural system.

13.5.3 Exterior Ornaments and Appendages

Exterior ornaments and appendages are nonstructural components that project above or away from the building. They include marquees, canopies, signs, sculptures, and ornaments, as well as concrete and masonry parapets. These components are acceleration sensitive, and if not properly braced or anchored can become disengaged from the structure and topple. Building codes require consideration of vertical accelerations for cantilever components. Features such as balconies are typically an extension of the floor structure, and should be designed as part of the structure. Parapets and cornices, unless well braced, are flexible components and design forces for these components should be amplified accordingly.

Heavy roof tiles pose a significant falling hazard, unless the tiles are securely attached to the roof diaphragm. One method of method of securing mission tiles is shown in Figure 13-11. The tie wires used to secure the tiles should be corrosion resistant.

13.5.4 Partitions

Partitions are vertical non-load bearing elements that are used to divide spaces. They may span vertically floor to floor or horizontally between cross walls. In some cases, partitions span to a hard ceiling (plaster or gypboard), or may extend to the ceiling, and stop, with lateral bracing extending to the floor or roof structure above. Partitions may classified as heavy or light. Heavy partitions are generally constructed of masonry materials including glass block masonry. They are self-supporting for gravity, isolated from the structural framework, and weigh in excess of 10 pounds per square foot (note that if these partitions are not isolated from the structural framework, they may behave as part of the buildings lateral force resisting system). Light partitions consist of wood or metal studs covered with gypboard, lath and plaster, or wood. Light partitions typically weigh less than 10 pounds per square foot.

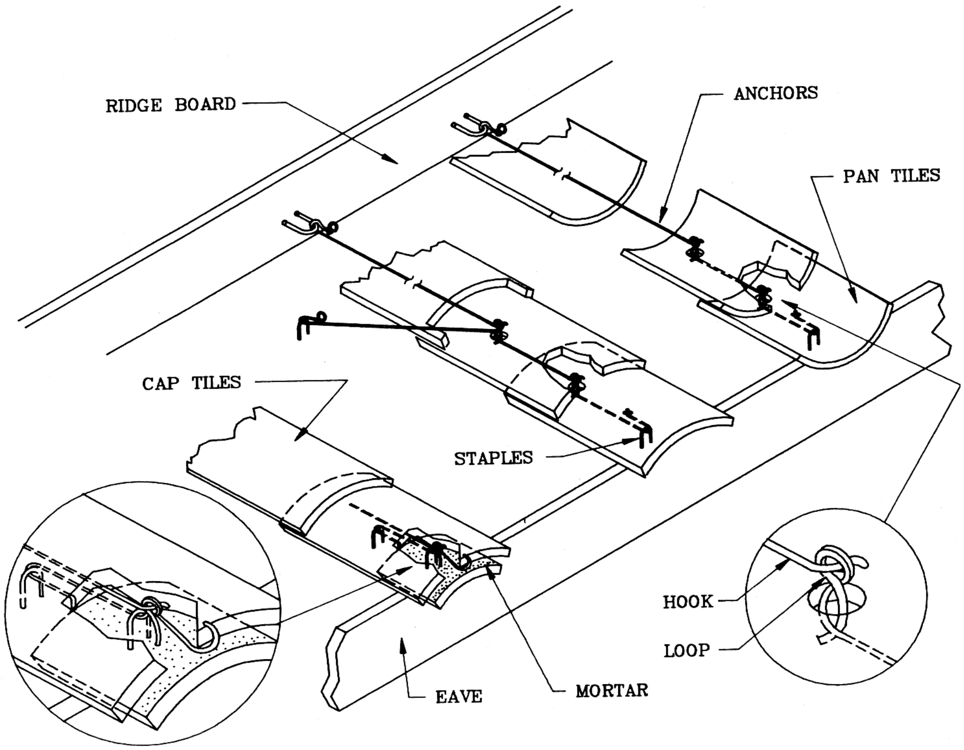


Figure 13-11. Roof tile anchor details

Partitions are acceleration and deformation sensitive. Partitions spanning floor to floor will suffer shear cracking and distortion due to story drift, unless detailed to accommodate drifts without racking. If the partitions undergo significant distortions, adhered veneers can fall off. In the out-of-plane direction, high accelerations can cause flexural cracking and if the top or bottom connections to the structure fail, collapse. If partitions are isolated from the supporting structure or are free standing, they became acceleration sensitive.

Seismic performance of partitions is controlled by attachment of the finish materials, and the support conditions at the floor or roof structure and ceiling system. The top connection should allow for vertical movement of the floor or roof structure and horizontal in-plane motion, but resist out of plane forces. Partitions in buildings with flexible structural frames should be anchored to only one structural element, such as a floor slab, and separated by a physical gap from all other structural elements. Reinforced masonry partitions tied to more than one structural element should be considered part of the structural system. Unreinforced masonry should not used for partitions or filler walls.

Connections at the top of the partition should accommodate in plane movement, but provide out of plane support. A gap, with an adequately sized resilient filler (if necessary for sound or fire separation), should isolate the structural frame from the nonstructural partition walls. Figure 13-12 illustrates one method of providing this separation for heavy partitions, while at the same time bracing the wall against out-of-plane motion. Figures 13-13 and 13-14 illustrate methods of bracing full and partial height light partitions. Partial height partitions should never be laterally supported by suspended T-bar ceiling systems. Partitions that cross building seismic joints are particularly susceptible to damage due to differential structural movement across the joints.

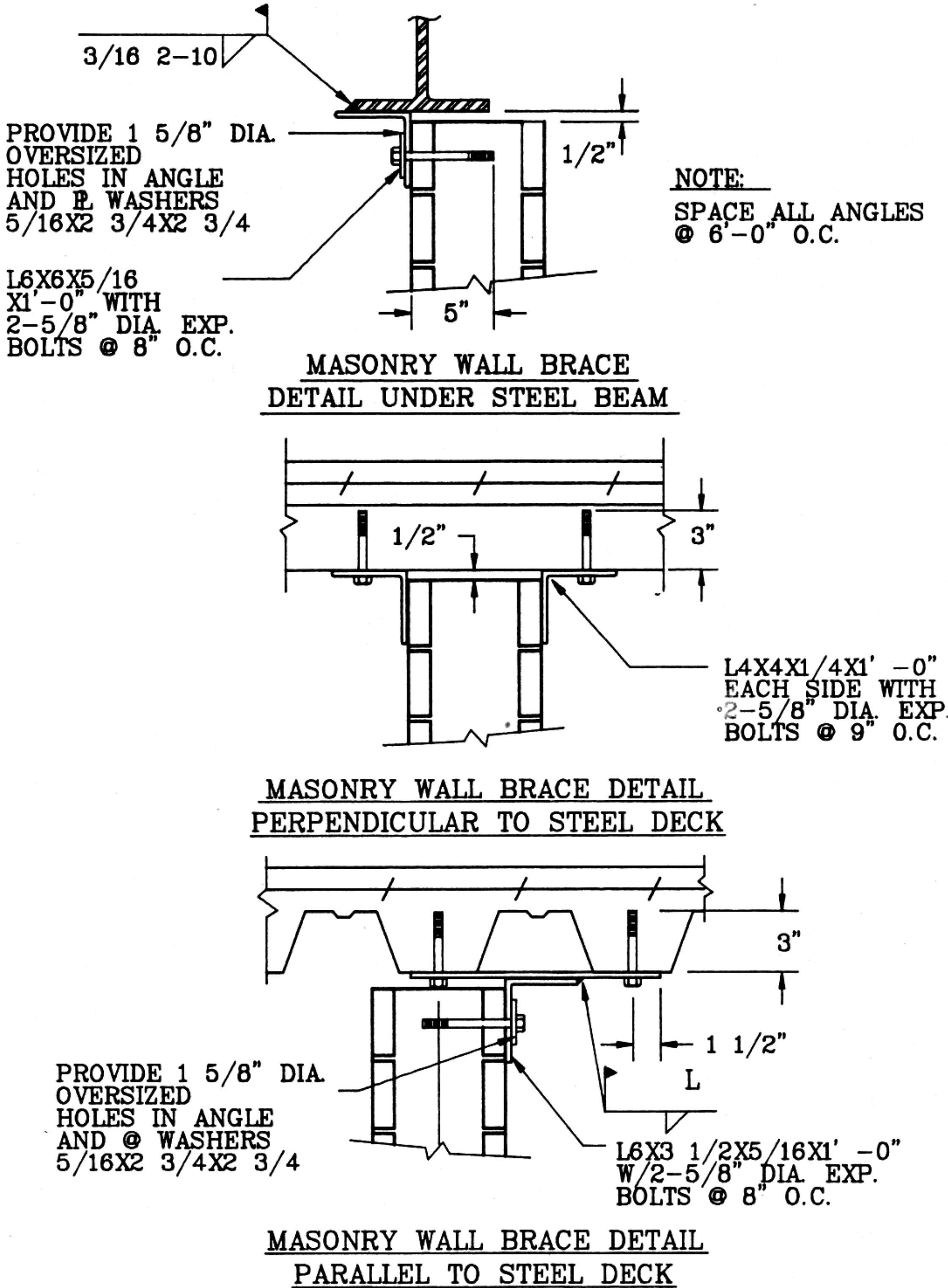


Figure 13-12. Nonbearing masonry wall details

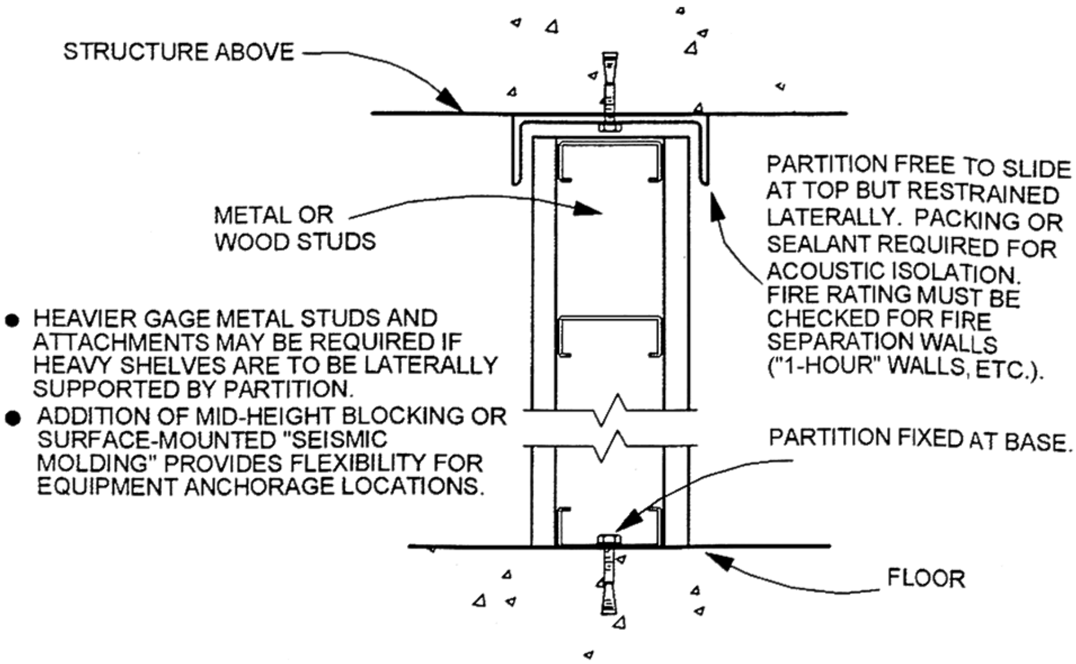


Figure 13-13. Seismic bracing for light partitions (13-10)

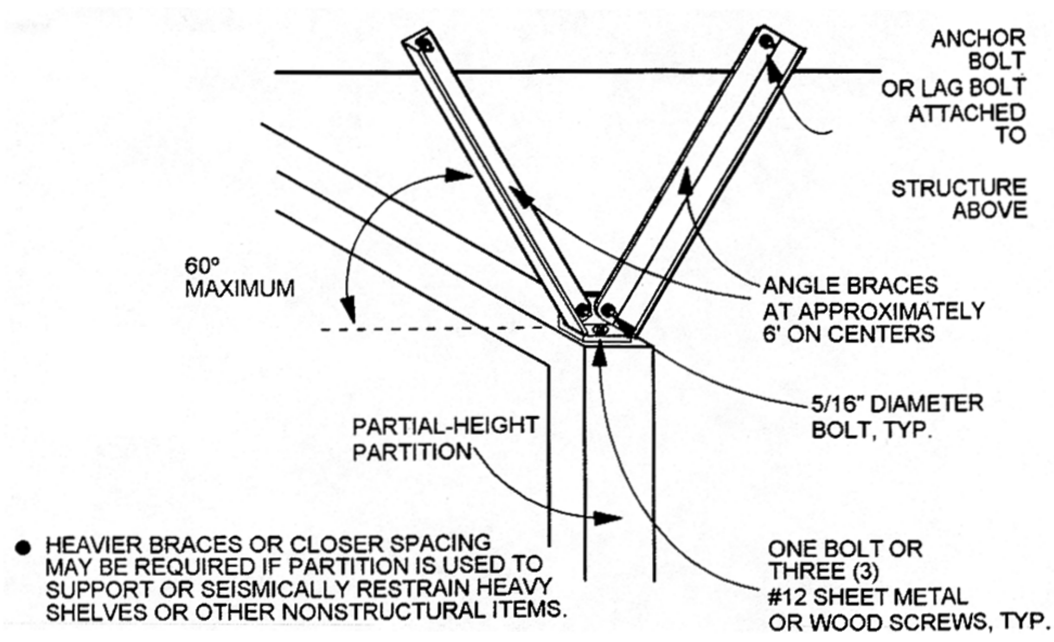


Figure 13-14. Seismic bracing for light partitions

Some modern interior planning approaches utilize nonanchored partition systems that rely upon the self-weight of the partitions, corners, or spread bases to supply stability. When subjected to seismic forces, these partitions are more susceptible to overturning than anchored systems. Decisions on their use should take into account the flexibility of such systems and their ease of installation, as opposed to the possible danger of overturning during an earthquake. Of particular importance in this regard are those systems that utilize hanging furniture or storage systems as part of the partition system. Some partitions are lightweight screens and may not necessarily cause injury or significant damage if overturned. However, other systems are more dense and heavier than full-height stud and gypsum board walls. The weight and stability of these systems must be given careful consideration in areas of high seismic risk.

13.5.5 Curtain Wall Systems

Curtain wall systems consist of prefabricated wall units and a variety of glass wall systems. Prefabricated wall systems include precast units (including units faced with an adhered or attached veneer), laminated metal faced insulating panels, unitized curtain wall systems, and steel framed panels with mechanically attached masonry, Glass Fiber Reinforced Concrete (GFRC), metal, or stone facing. These units may span vertically, from floor-to-floor or horizontally. Glass curtain wall systems include stick-framed systems assembled on site, sloped glazing and skylights, storefront systems, and structural glazing. Curtain wall systems are both acceleration and deformation sensitive, and can be dislodged by direct acceleration or failure of connections due to story drift.

Cladding units should have a minimum of four anchors per unit. Because of thermal movement and shrinkage considerations, the cladding unit connection system is generally statically determinate. Therefore, failure of a single connection can result in the cladding unit becoming unstable and falling from the building. The consequences of an anchorage failure are potentially grave, since large precast cladding units can weigh in excess of 20 kips. Therefore, building codes require that the connections for prefabricated panels be design for the unreduced expected story drift of the structure, and be able to resist high inertial loads in such a manner as to preclude failure. For precast concrete panels, connections are designed to ensure ductile behavior. The body of the connection, made up of steel plate or shapes, is designed for 1.33 times the design force for the panel skin. Elements of the connection that may behave in a brittle manner, such as welds, bolts, and items embedded in the concrete such as inserts and anchor bolts, are designed for 4 times the panel design force.

For panelized cladding systems, the units must be detailed in such a manner so as to permit lateral story drift of the structural frame. Units that span from floor to floor must accommodate drift through sliding or bending connections, or by rocking. Sliding connections may be detailed using bolts that slide in slotted holes. The length of the slot should equal twice the expected story drift, plus the diameter of the fastener, plus an allowance for construction tolerances. For sliding connections to be effective, the fastener should be centered in the slot, because if the bolt “bottoms out” at the end of the slot in an earthquake, extremely high shearing forces will be developed in the fastener. Connections that rely on bending in ductile elements can provide excellent performance, provided that the bending element is long enough to accommodate the expected story drifts without inelastic bending in the strain hardening range. Connections using threaded rods should be carefully designed, since the rod may suffer a low-cycle fatigue failure if subject to even moderate inelastic bending. The rocking mechanism permits cladding units to accommodate story drift by allowing vertical motions in the gravity load-bearing connections, through the use of vertical slots or oversize holes.

Special consideration should be given to the layout of joints in prefabricated wall systems.

At building corners, and when adjacent units utilize different methods for accommodating story drift, adjacent cladding units may not move in a uniform manner. Joints between cladding units may close causing adjacent panels to come into contact, imposing high loads on the panels and their anchors.

Glass curtain wall systems are typically assemblies of structural subframes attached to the main structure. They may be prefabricated or assembled on site, and include stick framed curtain walls assembled on site, prefabricated unitized curtain wall systems, storefronts, and skylights. Glazing systems are predominantly deformation sensitive, but can be damaged by high accelerations. Glazing in "dry" installations (where the glass is held in place by putty, a rubber/vinyl bead, or wood or metal stops) can shatter due to a combination of racking of the frame due to story drift coupled with out of plane forces. Failures of glazing systems in past earthquakes have been attributed to number of causes. Deficiencies in the design of the supporting frame and the cutting and placement of the glass can result in poor performance. A lack of sufficient support around the edges of the glass pane (edge bite), due to an oversized opening in the frame, or an undersized glass pane, may allow the glass to fall out. A lack of edge blocking can also allow the glass panes to shift and fall from the frame. If the glass panes are cut large, there may be insufficient clearance between pane and frame to permit racking. Frames that are attached to the structure that are not detailed to accommodate story drift will flex and twist. When frame racks due to story drift, the pane comes into contact with the frame and the glass, which cannot distort in-plane, will shatter. If the gasket around a glass pane loosens and falls from the opening, it may allow the glass to fall out, or move and shatter in the frame.

The type of glass used also affects safety. Ordinary annealed glass produces sharp-edged shards when broken. Safety (or tempered) glass is required when glass extends to within 18 inches of the ground or floor. Tempered glass fractures into small round-edged pieces, which pose a lower hazard. Laminated glass generally remains intact, even if it cracks. Tempered or laminated glass should be used in exits or where large glazed areas front public walks.

13.5.6 Ceiling systems

Ceiling systems are horizontal and sloping assemblies attached to or suspended from the structure. At exterior locations, ceiling systems may be referred to as soffits. While there are many different architectural treatments for ceilings, structurally, they can be classified into two main categories of systems, those that are attached directly to the building structure (surface applied materials), and those that are suspended from the structure by wires or other means.

Surface applied materials consist of wood, acoustical tile, gypboard, plaster, or metal panels applied directly to wood or steel joists, concrete slabs, or metal deck. The surface materials may be attached with mechanical fasteners or adhesive. This class of ceiling systems also includes gypboard ceilings attached to wood or steel furring supported by a supplemental framework, braced back to the primary structure. Surface applied materials typically perform well in earthquakes, provided the structural elements supporting this system perform reasonably well.

Suspended ceilings include T-bar systems with integrated lighting and mechanical components, and suspended lath and plaster or gypboard systems. There are a variety of suspended T-bar systems available. Most common are exposed spline systems (where the supporting T-bar frame is visible), concealed spline systems (hidden supporting frame), and luminous systems (lighting diffused through opaque panels). Suspended ceiling systems are acceleration and deformation sensitive. Seismic performance of suspended ceiling systems is controlled by the behavior of the support system, and historically concealed spline systems have performed better than exposed spline systems.

Suspended T-bar systems consist of a lightweight grid, which supports ceiling panels, light fixtures, and HVAC diffusers. These systems are highly vulnerable to damage unless the grid is securely braced with splay wires or other bracing devices and vertical compression struts (Figure 13-15). In an earthquake, the ceiling is subject to forces from light fixtures and ceiling ventilation diffusers. Sprinkler heads projecting through the ceiling may damage the panels and supports. Suspended ceiling systems in buildings with long spans and flexible structural systems are at greater risk.

Distortion of the grid can result in a loss of panels and may cause light fixtures and diffusers to drop. However, light ceiling panels that weigh less than 2 pounds per square foot do not pose a life safety risk, and are generally more of a nuisance than a hazard. Heavy items such as light fixtures and HVAC diffusers should have an independent supporting system, unless the ceiling suspension system is designed to carry the added weight of the fixtures during an earthquake. Positive, mechanical connections should be provided to keep the object attached to the grid. In addition, heavy items supported by the ceiling system should be provided with safety wires, to prevent the items from dropping should they become detached from the supporting grid.

Standards for seismic bracing of ceiling systems have been developed(13-11). In general, a seismic bracing strategy for suspended ceiling system should provide bracing against lateral and vertical movements. The disposition of bracing should account for concentrations of mass, such as light fixtures and diffusers. The ceiling system should be rigidly attached to two adjacent walls, and permitted to "float" along the walls directly opposite those where the system is attached. This permits the walls to distort and "rack" in plan without buckling the grid or pulling it apart. Along the "floating" edges, a shelf angle provides vertical support to the ceiling. The angle must be wide enough to

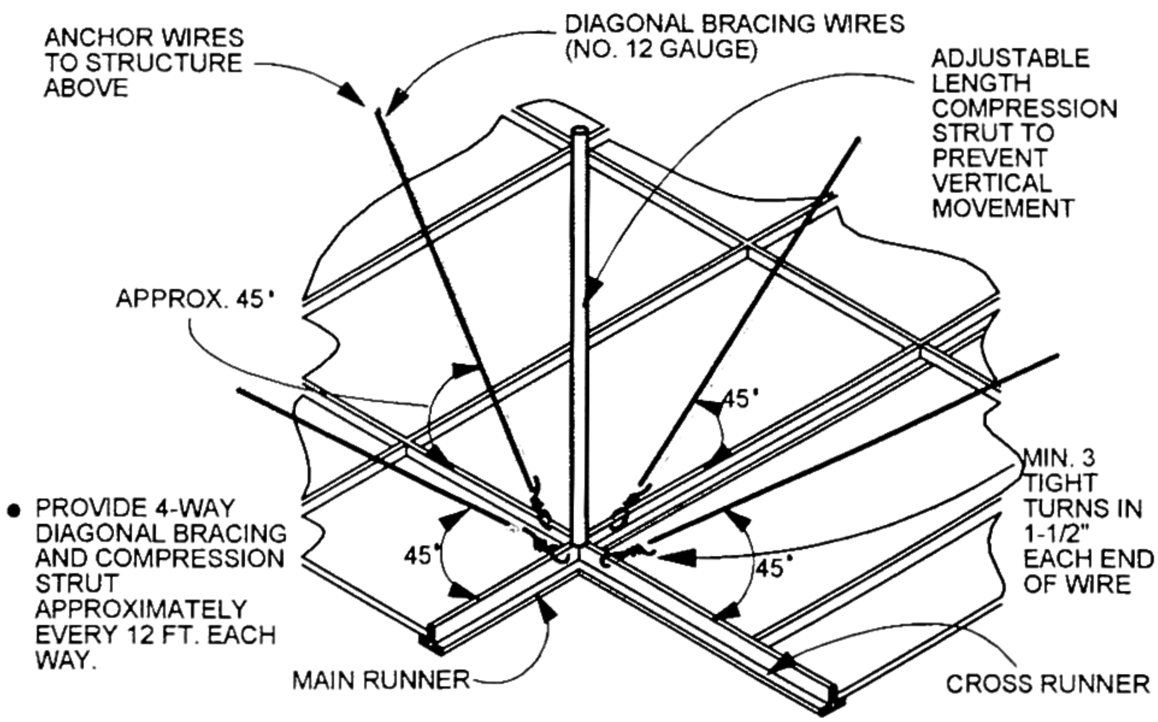


Figure 13-15. Seismic bracing for suspended T-bar ceiling systems (13-10)

allow for differential movement between opposing walls. Care must be taken to ensure that the grid is not inadvertently connected to the walls at the "floating" edge. At the perimeter of the ceiling, the main and cross runners of the ceiling grid should be supported by hangers.

The connection between the main runners and cross runners should positive, using locking clips or screws, to prevent the ceiling grid from coming apart during an earthquake. Friction-type connections should be avoided.

Suspended lath and plaster and gypboard systems can perform well, being inherently rigid in the plane of the ceiling. However, if the ceiling system is heavy and large, careful consideration of the design and detailing is needed, because the ceiling can pose significant risk to life if it drops. Reference 13-12 outlines the seismic design requirements for rigid suspended ceiling systems. Complex installations will require special engineering.

The hanger wires supporting the ceiling must be securely attached to the structure above, and the lath properly wired to the furring channels. Proper installation of these wires is crucial to satisfactory performance. Hanger wires may unwind, break, or fail at the connection to the structure in a strong earthquake. Isolation joints must be provided at building seismic separations. Without these joints, relative movements between the structures will damage the ceiling, and in some cases, collapse of the ceiling.

Rigid ceiling systems should be braced against vertical and lateral movement at regular intervals. Where gypsum board is used, seismic performance can be enhanced by the use of steel nailing strips. If the ceiling is irregular (with changes in elevation, reentrant corners, etc.), the supporting channels should be mechanically connected with bolts, screws, or welds. Corners should be rigidly braced. The arrangement of lateral bracing should consider discontinuities in the ceiling created by rows of light fixtures or HVAC diffusers. It may be desirable to add bracing members at these discontinuities, to tie the ceiling together. Light fixtures and diffusers should be securely fastened to the ceiling supports. The use of toggle bolts in the plaster or gypboard for attachment of these items should be avoided.

13.5.7 Exitways, Stair, and Elevator Enclosures

Exitways, stair, and elevator enclosures include treads, risers, landings, and surrounding shafts that make up the enclosures. These enclosures can be either acceleration or deformation sensitive. If integral with the structure, stairs and enclosures must be considered in the overall design and analysis, including their contribution to overall structural stiffness and response due to bracing action. Failure of the enclosure can render the stairs or elevator unusable.

Following an earthquake, building occupants will attempt to leave the building through the exitways. Care should be used, to ensure that design features of the exitways do not impeded safe egress. The doors should be designed to accommodate seismic drift, so they will not jam open or closed in an earthquake. The use of veneer or ceiling treatments that could become dislodged and fall should be avoided. The covers over seismic joints should be designed to accommodate the expected story drifts without significant damage. Light fixtures should be adequately braced for seismic loads. Stone veneers should be properly anchored to the supporting frames, and the frames should be designed to accommodate story drift without racking.

13.5.8 Building Contents

Building contents can pose a significant risk during a strong earthquake. The following section provides general information on improving the seismic performance of building contents. However, since the contents are generally furnished and installed by the owner, anchorage of these items is typically outside the scope of the design professional.

Storage racks, such as those found in warehouse stores, can pose a significant hazard. Storage racks installations should be engineered. Storage racks designed and installed in accordance with the standard of the Rack Manufacturers Institute[13-15] have been proportioned to withstand seismic forces. Special care should be taken to protect the legs of the racks, which are vulnerable to damage from forklifts. Adequate clearance between the rack and structural elements, such as walls and columns, should be provided to prevent interaction between the rack and the building structure.

Storage cabinets, bookshelves, filing cabinets, and display cases come in a myriad of shapes and sizes. In general, items that are tall and slender should be anchored to the wall to prevent tipping. Tall furniture, and items that have glass shelves should not be placed in the path of egress. Items that may shift or topple should not be placed where they could block exit doors. Providing latches on cabinet doors, and shelf lips or face bars on open shelving, can prevent loss of contents during ground shaking.

13.6 MECHANICAL/ ELECTRICAL COMPONENTS

13.6.1 General

Mechanical and electrical components consist of equipment such as pumps, boilers, chillers, fans, transformers, and electrical switchgear, as well as distribution systems such as piping, ducts, conduits, and cable trays. Most electrical and mechanical equipment are pre-manufactured, "off the shelf" items. The characteristics of each component are developed based upon functional needs. These characteristics – such as the presence of internal spring isolators or ceramic components, determine damage potential.

When discussing seismic performance of mechanical components, it is important to differentiate between Life Safety and Immediate Occupancy performance objectives. Functionality of the component following an earthquake is not generally a life safety issue. For the Life Safety objective, it is usually sufficient if the component does not shift or topple during an earthquake. For Immediate Occupancy, the component or system may be required to function following the earthquake. For the higher performance objectives, the component manufacturer must show through analysis or by shake-table test that the component remains functional following the prescribed level of ground shaking.

Much of the information in the following sections has been adopted from the discussion of mechanical and electrical components found in FEMA 273 and FEMA 274.

Mechanical and electrical equipment is generally acceleration sensitive. Failure modes include sliding, overturning, or tilting of items mounted on the floor or roof. Items suspended from or attached to walls or ceilings may suffer loss of support and fall. Distribution systems, such as piping, ducts or wiring connected to the unit can fail. Most equipment items are fairly robust, since they must survive the rigors of transportation and installation at the job site. However, the internal components of equipment may be blocked or restrained to prevent damage during transit. Upon removal of these restraints, the internal components of the item may be much more vulnerable to shaking damage.

Mechanical equipment and systems are either rigidly anchored to the primary structure, or installed on flexible mounts (to control vibration or permit thermal movements). The lateral capacity of rigidly mounted equipment is often governed by the capacity of the anchor bolt or fastener. Failures can also occur at the connection between the component and pipes, ducts, or conduits that connect to the component.

Vibrating mechanical equipment (typically equipment with rotating components, such as chillers, pumps or emergency generators) is often installed on resilient mounting systems, particularly when the equipment is on the upper floors of a structure. The most common vibration isolation mounts rely on springs or elastomeric devices to limit the transmission of vibration and sound to the rest of the structure. Unless specifically designed to resist seismic forces, isolated components are vulnerable to damage at low levels of ground shaking.

13.6.2 Rigidly Mounted Components

The primary aim of seismic design for rigidly mounted components is that they remain in place. The effects of shaking on internal parts are generally not considered. If functionality of the component is critical, then a special evaluation is required. This may include seismic qualification of the internal components of the equipment through shake table testing, detailed analysis, or experience data from past earthquakes. In this section, our focus is on anchorage issues, and the design of structural components of the equipment, such as base plates, anchor bolts, legs, braces, etc. Most equipment is not a life safety threat unless it can overturn or fall, or if failure of the component results in the interruption of a critical function or the release of hazardous materials.

The lateral capacity of rigidly mounted components can be governed by the capacity of the anchor bolts, the capacity of the unit frame or body, or by the capacity of a yielding element, such as a base plate or mounting tab. Installations with capacity governed by the anchor bolt capacity are the least desirable, but are often unavoidable. An example of a tank installation governed by anchor bolt capacity is illustrated in Figures 13-2 and 13-16. This rooftop saddle mounted tank displaced in the Northridge Earthquake. The tank itself was

Figure 13-16. Tank in figure 13-2, expansion anchor failure

undamaged. Introduction of a yielding element in the connection could have limited the loads delivered to the anchor bolts, precluding this failure. By introducing a yielding element, such as a steel plate in weak axis bending, the designer can introduce a yielding element, providing a mechanism for the dissipation of energy and limiting the amount load that can be delivered to the anchor bolts. Where this cannot be done, the design forces for the anchor bolts should be increased, to preclude a brittle failure of the installation.

The designer should consider the load path for the seismic forces of the component. The design anchorage forces will generally be significantly higher than the design lateral forces for the primary structure or framing supporting the component. For example, a heavy air conditioning unit may be mounted on a roof diaphragm of untopped steel deck. At the point of attachment, the air conditioning unit may deliver design lateral forces to the steel deck that exceed the design shears for the roof diaphragm. The designer should check the load path – in this case, the diaphragm capacity in the immediate vicinity of the unit – for the component vertical and lateral loads. Local elements of the supporting structure should be designed and constructed for the component forces, where they control the design of the elements or their connections. When checking the supporting structure, the design forces should not be modified due to anchorage conditions. For example, using the 1997 NEHRP provisions, it would not be necessary to reduce the R_p factor due to shallow anchor bolt embedment.

In general, installations that rely on threaded pipe connections, for example, a vertical tank supported on pipe legs, should be avoided. Threaded connections are subject to low-cycle fatigue failures. Saddle mounted tanks should be restrained by straps or lugs to the supporting frame. Pipe and conduit connections to equipment should be designed to accommodate differential movement, through the use of braided or flexible connections.

13.6.3 Vibration Isolated Components

A vibration-isolated component can experience much higher seismic accelerations than the same component, rigidly mounted. This is due to the amplification effects of the vibration mounts. The dynamic characteristics of vibration-isolated equipment are dominated by the properties of the isolation mount. The fundamental period of isolated components can lengthen to the point were a resonance condition with one or more modes of the primary structure is possible. This can result in amplifications in lateral force by a factor of five or more. The key to controlling these effects is through the use of snubbers.

Isolated components can either be internally snubbed (the snubber is an integral part of the vibration isolation device) or externally snubbed (through the use of separate snubbers, installed independent of the isolation device). Regardless of the type of snubber used, it is vital that an elastomeric pad be provided to reduce the impact force generated when the component strikes the snubber. Selection of the proper elastomeric pad can be crucial, and the manufacturers' recommendations for the material should be closely followed. Research and experience has shown that the degree of force amplification due to impact can be reduced if the clearance between the component and the snubber (air gap) is limited to ¼ inch or less. The use of inertia pads above the vibration isolators should be carefully considered in the design of the system, since they can add a great deal of mass. As with fixed components, it is vital that a load path of adequate strength be provided for vibration-isolated components. Special attention should be given to the reinforcement of housekeeping slabs, and to their connection to the structural slab. Ideally, the housekeeping slabs should be cast monolithically with the structural slab. If this is not possible, sufficient dowels should be provided to transfer the lateral forces from the component to the diaphragm. The design lateral force should include any amplification effects due to the vibration isolators, and friction due to gravity forces should be neglected.

When suspended components are mounted with vibration isolation devices, care must be taken to ensure that the bracing elements do not "short out" the vibration isolators. For example, the benefits of vibration isolation may be lost if the unit is laterally braced with steel angles to the structure. The hangers and braces must be designed for the amplified forces, and if hanger rods are used, the may need to be stiffened to prevent buckling under the compressive loads generated by the vertical component of the brace force. If the body of the component does not have sufficient strength and rigidity, a supplemental structural frame around the item may be necessary.

Flexible couplings should be provided where pipes, ducts, or conduits meet vibration-isolated systems. Figure 13-4 illustrates the results of vibration-isolated component rigidly attached to a braced pipe.

13.6.4 Piping Systems

Piping systems are predominantly acceleration sensitive, but runs between floors or buildings are deformation sensitive. Joint failures caused by inadequate support or bracing, with accompanying loss of contents under pressure, are the most common failures. Most pressure piping systems (defined as piping systems carrying fluids which, in their vapor stage, exhibit a gage pressure greater than 15 psi) are inherently ductile and have sufficient inherent flexibility to accommodate seismic motions. Attachments and braces for seismic loading are needed, particularly for large diameter pipes. Bracing in most installations is performed to prescriptive standards, such as the SMACNA and NFPA-13 guidelines[13-13,13-14].

Flexible couplings to accommodate building movements should be provided at structural separations, as well as at the base of the structure where pipes pass from the ground into the structure. Figure 13-17 illustrates such an

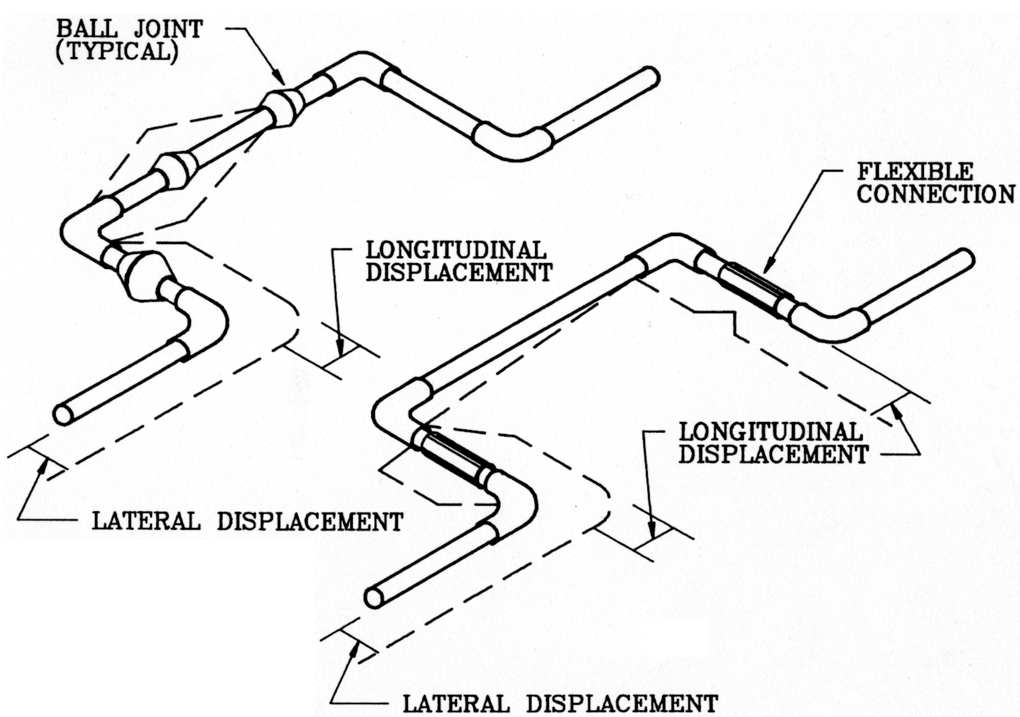


Figure 13-17. Piping details at a seismic gap

arrangement.

Damage to fire suppression piping has generally been the result of joint failures and differential movement between the piping and portions of structure. Failures have been caused by impact of branch lines and sprinkler heads on adjacent elements, such as hard ceilings. Providing sway bracing at fire sprinkler branch lines and long sprinkler drops can reduce this type of damage. Providing larger openings for the sprinkler heads in hard ceiling surfaces can prevent the ceiling from fracturing the sprinkler heads due to movement of the piping.

Sway bracing requirements for piping are specified in building codes and bracing is generally required at specified intervals, based on the size of the pipe. A typical pipe brace installation is shown in Figure 13-18. Additional bracing should be provided at bends and elbows. Flexible couplings should be provided where piping crosses seismic separation joints, and where the piping is connected to vibration isolated equipment. Small diameter pipes that are allowed to sway should have flexible couplings installed at equipment connections. Where piping penetrates walls or floors, the pipe sleeves should be large enough to accommodate any anticipated relative movements.

The designer should note that in general, the sway bracing specified in the prescriptive standard may not prevent local leaks. If the contents of the piping system are hazardous, a more detailed analysis is warranted.

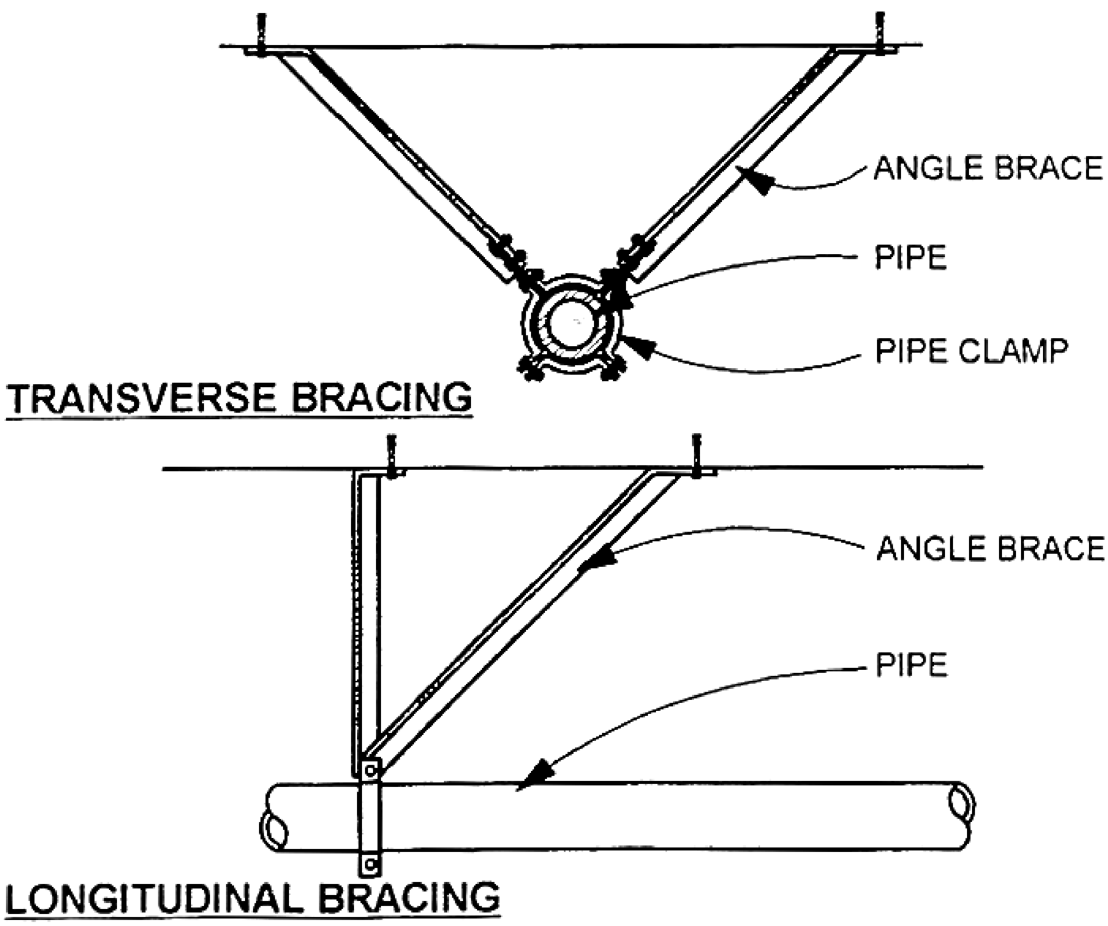


Figure 13-18. Seismic sway bracing for piping systems (13-10)

13.6.5 Air Distribution Systems

Sheet metal ducts can tolerate large distortions and generate low inertial loads, but have little inherent strength. Ducts rarely collapse, but the joints are particularly vulnerable. Joint failures result in a loss of air pressure. In line equipment such as axial flow fans should be braced independent of the duct system.

Air distribution systems are predominantly acceleration sensitive, but runs between floors or buildings are deformation sensitive. Large ducts (over six square feet in area or 24 inches in diameter) should be braced to the structure. Figure 13-19 illustrates a method of providing this bracing. Flexible duct connections should be installed with enough slack material to allow for the expected differential movement between fans and the ductwork. Duct openings through walls or floors must be large enough to accommodate the anticipated movement of the ducts. Ceiling diffusers and registers should be secured to ductwork with sheet metal screws, to prevent them from falling should they become dislodged. Diffusers connected to flexible ducts should be provided with safety wires to the floor or roof above, and should be securely fastened to the ductwork.

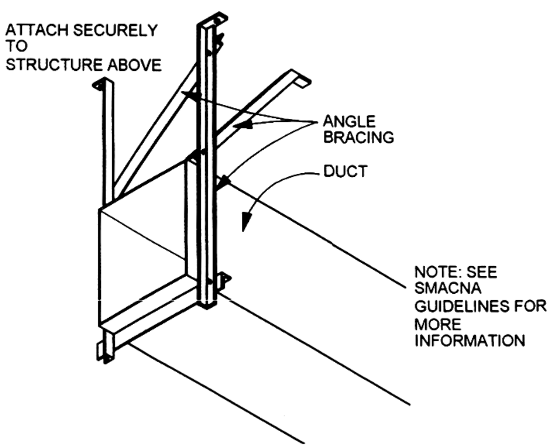


Figure 13-19. Seismic sway bracing for HVAC duct systems [13-10]

13.6.6 Elevators:

Elevator equipment includes the mechanical equipment such as motor generators and sheaves, electrical controllers, as well as the car and counterweight frames and guide rails. Elevator machinery behaves in the same manner as other heavy floor-mounted equipment. Mechanical and electrical components should be anchored to resist inertial forces.

Experience in past earthquakes has shown that the counterweight rails in elevators are vulnerable to damage. During the shaking, the rails can bend, allowing the counterweights to displace into the elevator shaft. Unless a careful post-earthquake survey of the shaft is made, it may not be apparent that this has occurred. The danger is that the displaced counterweights may strike the car if the elevator is operated. Recent editions of the building codes have required that the heavier counterweight guide rails be used, to limit distortions in an earthquake. Elevators should be equipped with a seismic switch, that senses significant ground shaking and shuts the elevator down, or forces it to operate in a “go slow” mode. The seismic switch should only be reset after an inspection by a qualified technician.

13.6.7 Electrical Equipment

Electrical equipment includes electrical and communication equipment, electrical panels, motor control centers, switch gear, transformers, emergency generators, battery racks, light fixtures, and other fixed components, as well as distribution systems such as conduit and cable trays. These components are generally acceleration sensitive, except for conduit and cable trays crossing building separations or running from floor to floor, which may deformation sensitive.

Electrical panels may be flush or surface mounted. Flush mounted panels generally perform well, providing the panel is attached to the wall studs that frame the opening for the panel. Surface mounted panels should be screwed or bolted into the supporting wall studs or into a steel backing plate that spans between the studs. Toggle bolts in plaster or gypboard should be avoided.

Motor control centers and switchgear should be anchored to the floor, and for tall units, braced or anchored at the top. If structural bracing at the top is omitted at tall units, conduit running into the upper portions of the unit may be damaged while acting as bracing. Where relative movement between units could occur, flexible braided connections should be used in lieu of copper bus.

Emergency generators are generally installed on vibration isolation mounts. In order to function after an earthquake, all components of the system should be anchored for seismic forces. These components include the prime mover and generator, starting equipment including batteries, day tank, main fuel tank, radiators, exhaust silencers/mufflers, as well as the motor control and switchgear. If the generator is mounted on vibration isolators, the line from the main fuel tank to the generator should be flexible enough to accommodate the expected lateral displacements of the isolated components. Emergency generators that rely upon the municipal water supply for engine cooling should be avoided, since utility lifelines generally fail following a strong earthquake. Batteries should be restrained in adequately anchored racks or boxes.

Transformers come in a variety of sizes, and can be floor mounted, or suspended from the walls or ceiling. If wall or ceiling mounted, the transformers should be adequately attached to the supporting frame, and the frame proportioned to resist seismic forces. In some cases, floor mounted transformers are stacked, to conserve space. If the units are stacked, the upper unit should be bolted to the lower unit, and the adequacy of the lower unit frame or enclosure should be verified for the anchorage forces at the connection between the two units.

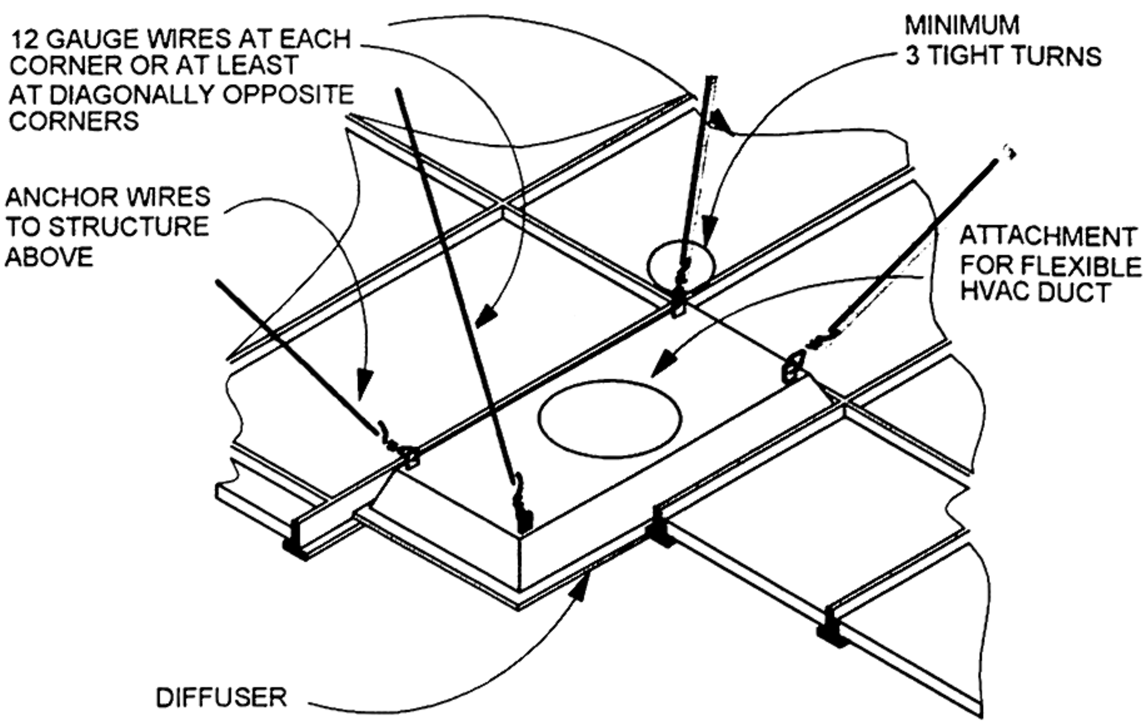


Figure 13-20. Seismic safety wires for HVAC diffusers, light fixtures

Lighting fixtures come in a number of types, and may be recessed and surface mounted in ceilings or walls, supported within a suspended ceiling, and suspended from the ceiling or structure (pendant fixtures). Lighting fixture support failures are generally due to failure of the attachment of the fixture to the wall or ceiling, or failure of the wall or ceiling. Distortions of T-bar system may allow fixtures to fall. Providing slack safety wires at opposing corners of light fixtures in T-bar ceilings will prevent them from falling, should the grid distort and the fixtures become detached. Excessive swing of pendant fixtures should be avoided, since it may result in impact with other building components or the support attachment may pull out of the ceiling. Recessed lighting fixtures should be secured to the ceiling suspension system. The suspension system should be of intermediate or heavy grade construction, and be designed to carry the weight of the ceiling fixtures. In addition, the fixtures should be provided with independent safety supports (Figure 13-20). Auxiliary support framing is required where the alignment of the lighting fixtures concentrates significant mass in a portion of the ceiling grid.

Conduits and cable trays should be braced at regular intervals. Where conduit and bus ducts pass across seismic joints, flexible connections that can accommodate the expected relative displacements should be provided. Separate ground connectors should be provided in conduit runs that pass across seismic joints.

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