Related Commercial Resources

CHAPTER 54

SEISMIC AND WIND RESTRAINT DESIGN

ALMOST all inhabited areas of the world are susceptible to the damaging effects of either earthquakes or wind. Restraints that are designed to resist one may not be adequate to resist the other. Consequently, when exposure to either earthquake or wind loading is a possibility, strength of equipment and attachments should be evaluated for all appropriate conditions.

Earthquake damage to inadequately restrained HVAC&R equipment can be extensive. Mechanical equipment that is blown off the support structure can become a projectile, threatening life and property. The cost of properly restraining the equipment is small compared to the high costs of replacing or repairing damaged equipment, or compared to the cost of building downtime due to damaged facilities.

Design and installation of seismic and wind restraints has the following primary objectives:

- To reduce the possibility of injury and the threat to life.
- To reduce long-term costs due to equipment damage and resultant downtime.

Note: The intent of building codes with respect to seismic design is not to prevent damage to property or the restrained equipment itself.

This chapter covers the design of restraints to limit equipment movement and to keep the equipment captive during an earthquake or extreme wind loading. Seismic restraints and isolators do not reduce the forces transmitted to the restrained equipment. Instead, properly designed and installed seismic restraints and isolators have the necessary strength to withstand the imposed forces. However, equipment that is to be restrained must also have the necessary strength to remain attached to the restraint. Equipment manufacturers should review structural aspects of the design in the areas of attachment to ensure the equipment will remain attached to the restraint.

For mechanical systems, analysis of seismic and wind loading conditions is typically a static analysis, and conservative safety factors are applied to reduce the complexity of earthquake and wind loading response analysis and evaluation. The following three aspects are considered in a properly designed restraint system:

- *Attachment of equipment to restraint.* The equipment must be positively attached to the restraint, and must have sufficient strength to withstand the imposed forces, and to transfer the forces to the restraint.
- *Restraint design.* The restraint also must be strong enough to withstand the imposed forces. This should be determined by the manufacturer by tests and/or analysis.
- *Attachment of restraint to substructure.* Attachment may be by bolts, welds, or concrete anchors. The substructure must be capable of surviving the imposed forces.

The preparation of this chapter is assigned to TC 2.7, Seismic and Wind Restraint Design.

SEISMIC RESTRAINT DESIGN

Most seismic requirements adopted by local jurisdictions in North America are based on model codes developed by the International Conference of Building Officials (ICBO), Building Officials and Code Administrators International (BOCA), the Southern Building Code Conference, Inc. (SBCCI), and the National Building Code of Canada (NBCC), or on the requirements of the National Earthquake Hazards Reduction Program (NEHRP). The model code bodies, working through the International Code Council (ICC), have unified their model codes into the International Building Code (IBC). Local building officials must be contacted for specific requirements that may be more stringent than those presented in this chapter.

Other sources of seismic restraint information include

- *Seismic Restraint Manual: Guidelines for Mechanical Systems*, published by SMACNA (1998), includes seismic restraint information for mechanical equipment subjected to seismic forces of up to 1.0*g*.
- The National Fire Protection Association (NFPA) standards on restraint design are the same as those in the IBC.
- U.S. Department of Energy DOE 430.1A and ASME AG-1 cover restraint design for nuclear facilities.
- *Technical Manual* TM 5-809-10, published by the U.S. Army, Navy, and Air Force (1992), also provides guidance for seismic restraint design.
- *A Practical Guide to Seismic Restraint* [ASHRAE Research Project RP-812 (2000)], covers a broad range of seismic restraint design issues.

In seismically active areas where governmental agencies regulate the earthquake-resistive design of buildings (e.g., California), the HVAC engineer usually does not prepare the code-required seismic restraint calculations. The HVAC engineer selects the heating and cooling equipment and, with the assistance of the acoustical engineer (if the project has one), selects the required vibration isolation devices. The HVAC engineer specifies these devices and calls for shop drawing submittals from the contractors, but the manufacturer employs a registered engineer to design and detail the installation. The HVAC engineer reviews the shop design and details the installation, reviews the shop drawings and calculations, and obtains the approval of the architect and structural engineer before issuance to the contractors for installation.

To ensure that the proper design factors are used, a responsible source on the project team must provide information on the seismic design conditions (zone, soil type, etc. as applicable) to the engineer evaluating the installation.

Anchors for tanks, brackets, and other equipment supports that do not require vibration isolation may be designed by the building's structural engineer or by the supplier of the seismic restraints (depending on contractual arrangements between design team members), based on layout drawings prepared by the HVAC engineer. The building officials maintain the code-required quality control over the design by requiring that all building design professionals are registered (licensed) engineers. Upon completion of installation, the supplier of the seismic restraints, or a qualified representative, should inspect the installation and verify that all restraints are installed properly and comply with specifications.

TERMINOLOGY

Base plate thickness. Thickness of the equipment bracket fastened to the floor.

Effective shear force V_{eff}. Maximum shear force of one seismic restraint or tie-down bolt.

Effective tension force T_{eff}. Maximum tension force or pullout force on one seismic restraint or tie-down bolt.

Equipment. Any HVAC&R component that must be restrained from movement during an earthquake.

Fragility level. Maximum lateral acceleration force that the equipment is able to withstand. These data may be available from the equipment manufacturer and are generally on the order of four times the acceleration of gravity (4*g*) for mechanical equipment.

Resilient support. An active seismic device (such as a spring with a bumper) to prevent equipment from moving more than a specified amount.

Response spectra. Relationship between the acceleration response of the ground and the peak acceleration of the earthquake in a damped single degree of freedom at various frequencies. The ground motion response spectrum varies with soil conditions.

Rigid support. Passive seismic device used to restrict any movement.

Shear force *V*. Force generated at the plane of the seismic restraints, acting to cut the restraint at the base.

Seismic restraint. Device designed to withstand seismic forces and hold equipment in place during an earthquake.

Seismic force levels. The geographic location of a facility determines its seismic zone, as given in the Uniform Building Code or International Building Code.

Snubber. Device made of steel-housed resilient bushings arranged to prevent equipment from moving beyond an established gap.

Tension force *T*. Force generated by overturning moments at the plane of the seismic restraints, acting to pull out the bolt.

CALCULATIONS

Sample calculations presented here assume that the equipment support is an integrated resilient support and restraint device. When the two functions of resilient support and motion restraint are separate or act separately, additional spring loads may need to be added to the anchor load calculation for the restraint device. Internal loads within integrated devices are not addressed in this chapter. These devices must be designed to withstand the full anchorage loads plus any internal spring loads.

Both static and dynamic analyses reduce the force generated by an earthquake to an equivalent statically applied force, which acts in a horizontal or vertical direction at the component's center of gravity. The resulting overturning moment is resisted by shear and tension (pullout) forces on the tie-down bolts. Static analysis is used for both rigidly mounted and resiliently mounted equipment.

Dynamic Analysis

Dynamic analysis of the isolation and snubber systems may be based on ground-level response spectra given in both IBC 2000 and IBC 2003, or NFPA Code 5000 (referenced in SEI/ASCE *Standard* 7-02) can be used as input for a dynamic analysis. Upper-level response spectra can be found in ATC 29-2 or ICC-ES (2007). Site-specific ground response spectra developed by a geotechnical or soils engineer may be used, as well. The computer analysis used must be capable of analyzing nonlinear supports and site-specific ground motions. This dynamic analysis provides the maximum seismic input accelerations to the equipment components, allowing comparison to three-dimensional shock (drop) or shaker test fragility levels to determine equipment survivability. Actual drop or shaker test data for all HVAC equipment may not be available for the next several years. Table 1 provides minimum fragility level guidelines to help evaluate the equipment's survivability.

Table 1 Minimum Fragility Level Guidelines

Item	Acceleration, *g*: No Damage Probable	Acceleration, *g*: Minor Damage Probable
Pumps (centrifugal) up to 100 hp	4.5	8
Computer air-conditioning units	4.5	N/A
Variable-air-volume (VAV) boxes	3.5	8
Control panel	2	N/A
Fans (centrifugal and axial) up to 100 hp	4	9
Welded steel piping (up to 16 in. diameter), schedule 30 and up	4	8
Rooftop units, curb-mounted	3	6
Air-handling units (AHUs) to 63 ft^2 coil area	4.5	10
Duct		
Rectangular steel	4.5	9
Round steel	4.5	10
MCC motor control centers (MCC)	2.5	N/A
Branch circuit lighting and power panels	3.5	N/A
Uninterruptible power supply (UPS) system	3	N/A
System control cab	3.5	N/A
Battery racks	3.5	7
Substation switch gear	3.5	N/A
Substation transformers	2	N/A
Cable tray to 36 in.	4.5	8
Bus duct	4.5	8
Water heater	3	6
Cooling towers:		
Ceramic	2.5	4
Steel, 1 or 2 cells	4	8
3 cells or more	3.5	6
Wood	4	8
Chillers:		
Centrifugal	4	N/A
Absorption	4.5	N/A
Screw	3.5	N/A
Air-cooled	3.0	N/A
Boilers, fire tube	3.5	N/A
Air compressor, tank mounted		
to 5 hp	4.5	8
7.5 to 20 hp	4	7
Frame mounted, 20 to 100 hp	3.5	7

Sources: ASHRAE (2000), Cover et al. (1985), DOD (1990, 2002), Bulletin No. 32 Shock, Vibration and Associated Environments—Protective Construction Part III, Seismic Fragility based on Test Data—Takanori Ogata July 1992, Seismic Fragility Analysis of Structures and Components for HFBR Facilities BNL-46561, and Chapter 50 of the 1995 *ASHRAE Handbook—HVAC Applications.*

Using the response spectra in the code for ground-floor inputs, or the spectra in ATC 29-2 for upper floors, a dynamic analysis can yield maximum input accelerations to equipment components. Comparing them to the allowable acceleration values in the table helps the engineer assess equipment survivability. Dynamic analysis can also provide maximum movement at all connections and, when added to the floor-to-ceiling code-mandated movements, allows the engineer to design these flexible connections and avoid pull-out or shear failures at these locations.

Under some conditions, Chapter 17 of IBC 2000/2003 requires certificates of compliance for components and their attachments for a component importance factor I_p [see Equations (1)] of 1.0 or 1.5.

Table 2 IBC 2000/2003 Seismic Analysis Requirements

Seismic Use Group (Building)	Component Operation Required for Life Safety	Building Seismic Design Category*	Required Analysis Type: Anchorage	Equipment Structural Capacity	Equipment Operational Capacity	Certificate of Compliance
I, II, III	No	A	Not required	Not required	Not required	Not required
I, II	No	B, C	Not required	Not required	Not required	Not required
I, II	No	D	Static	Dynamic or test	Not required	For mounting only
I, II	Yes	C, D	Static	Dynamic or test	Dynamic or test	For continued operation
I, II	No	E	Static	Dynamic or test	Dynamic or test	For continued operation
III	No	C, D	Static	Not required	Not required	Not required
III	Yes	C, D	Static	Dynamic or test	Dynamic or test	For continued operation
III	No	F	Static	Dynamic or test	Not required	For mounting only
III	Yes	F	Static	Dynamic or test	Dynamic or test	For continued operation

*If in question, reference structural documents

Table 3 Coefficients for Mechanical Components (2000/2003 IBC and NFPA *Code* 5000)

Mechanical and Electrical Component or Element	a_p	R_p
General Mechanical		
Boilers and furnaces	1.0	2.5
Piping		
High-deformability elements and attachments	1.0	3.5
Limited-deformability elements and attachments	1.0	2.5
Low-deformability elements or attachments	1.0	1.25
HVAC Equipment		
Vibration isolated	2.5	2.5
Non-vibration isolated	1.0	2.5
Mounted in-line with ductwork	1.0	2.5

Source: International Building Code (2000).

This is a life safety issue as well as an essential equipment issue. Essential equipment with an $I_p = 1.5$ must have a certificate of compliance. Issuance of a certificate of compliance to the engineer of record and building official can be based on dynamic analysis. Most building officials require a stamp by a registered professional to be part of the calculations and certificate of compliance. Table 2 provides guidance on type of analysis (static or dynamic) and certificate of compliance documentation is required. Sample dynamic analysis is beyond the scope of this chapter and should be provided by experienced registered professionals. A common approach assumes an elastic response spectrum. The results of the dynamic analysis can then be scaled up or down as a percentage of the total lateral force obtained from the static analysis performed on the building.

Dynamic analysis of piping, ductwork, and equipment reflects the response of the equipment for all earthquake-generated frequencies. Especially for piping and equipment, when the earthquake forcing frequencies match the natural frequencies of the system, the resulting applied forces increase.

Establishing Design Static Force

This chapter details code requirements for the 2000 and 2003 IBC (International Building Code), the 1997 UBC (Uniform Building Code) and the 1994 UBC. The 2000 IBC is expected to become the single national building code for the United States; however, the 1997 UBC and its associated maps are not expected to be replaced internationally for some time.

Until new spectral maps are available for locations outside of the United States, seismic design forces for these areas should be determined based on the older seismic “zone” and ground motion data that are available. Because these data are not compatible with the 2000 IBC design equations, the design force should be determined based on criteria in the 1997 UBC.

To address the bulk of applications, design parameters used by all three versions of the design codes are discussed.

Static Analysis as Defined in the International Building Code

The 2000 and 2003 IBC and NFPA *Code* 5000 specifies a design lateral force F_p for nonstructural components as

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

but F_p need not be greater than

$$F_p = 1.6\, S_{DS} I_p W_p \tag{2}$$

nor less than

$$F_p = 0.3 S_{DS} I_p W_p \tag{3}$$

$$S_{DS} = 2F_a S_S/3 \tag{4}$$

where

a_p = component amplification factor in accordance with Table 3.

S_{DS} = design spectral response acceleration at short periods. S_S is the mapped spectral acceleration from Figure 1 and $0.8 \le F_a \le 2.5$. (*Note*: More detailed maps for the United States are available at the U.S. Geological Survey Web site: www.usgs.gov)

F_a = function of site soil characteristics and must be determined in consultation with either project geotechnical (soils) or structural engineer. Values for F_a for different soil types are given in Table 4. (*Note*: Without an approved geotechnical report, the default site soil classification is assumed to be site class *D*.)

R_p = component response modification factor in accordance with Table 4, IBC 2000/2003. (*Note*: If shallow-embedment post-installed anchors, shallow adhesive anchors, or shallow cast-in-place anchors are used, then $R_p = 1.5$. Shallow embedment is defined as $L/d < 8$.)

I_p = component importance factor = 1.5 if one of the following four criteria is met, otherwise 1.0: (1) life safety component required to function after an earthquake, (2) component contains higher than exempted amount of hazardous flammable material, (3) storage rack in occupancies open to general public, and (4) component is needed for continued operation of a Use Group III facility (Table 5).

$1 + 2z/h$ = height amplification factor where z is the height of attachment in the structure and h is the average height of the roof above grade. The value of $z \ge 0$ and z/h need not exceed 1.

W_p = weight of equipment, which includes all items attached or contained in the equipment

If the equipment being analyzed is isolated, the final computed force must be doubled per section 1621.3.1 of the code.

Fig. 1A Maximum Considered Earthquake Ground Motion for the United States

0.2 s spectral response acceleration (% *g*) (5% of critical damping) Site Class B

(Electronic version also available at the USGS web site http://geohazards.cr.usgs.gov/eq)

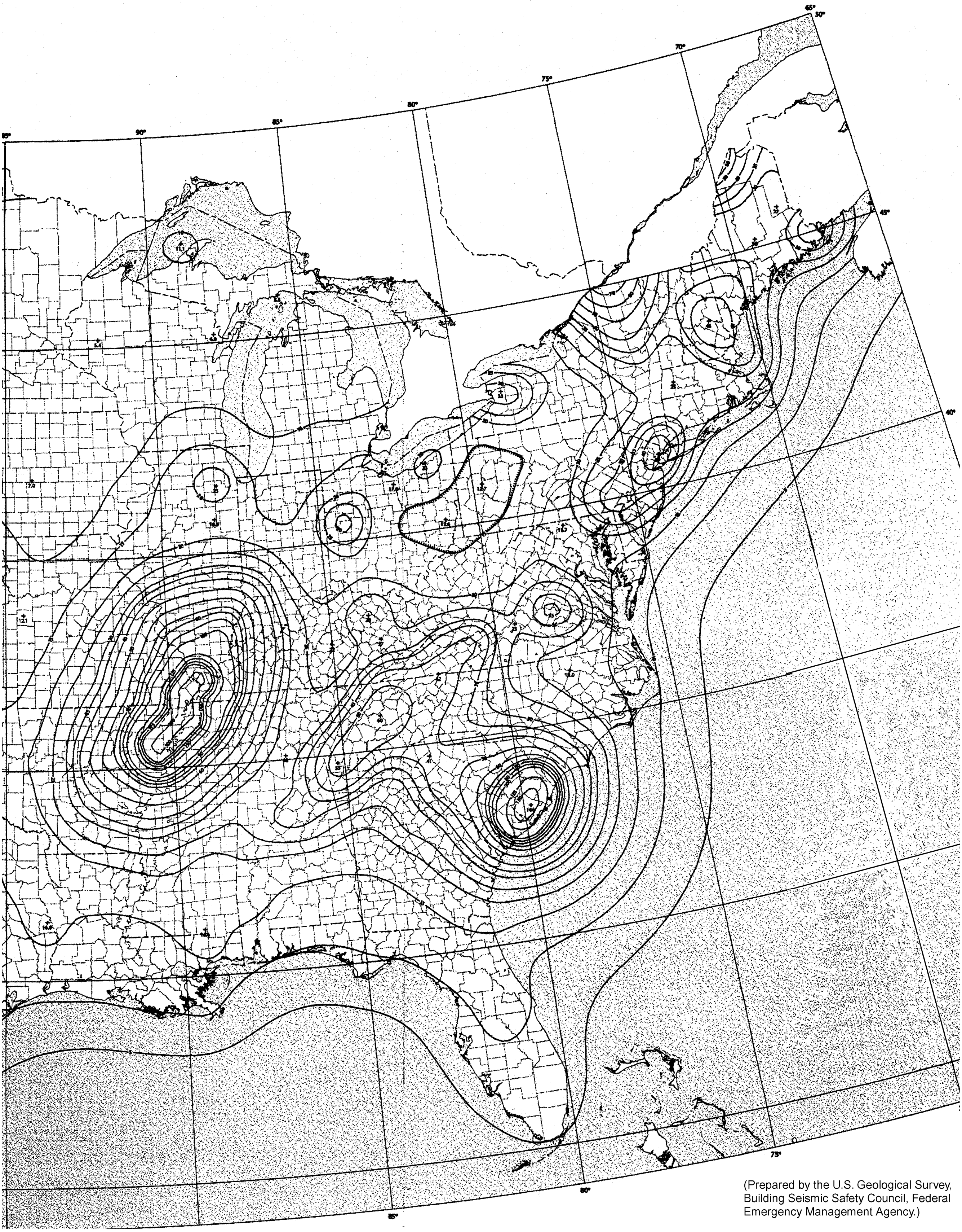

Fig. 1B Maximum Considered Earthquake Ground Motion for the United States (*Concluded*)
0.2 s spectral response acceleration (% *g*) (5% of critical damping) Site Class B
(Electronic version also available at the USGS web site http://geohazards.cr.usgs.gov/eq)

Per section 1621.1.7 of the code, forces used when computing the loads for shallow (under 8 bolt diameter) embedment anchors are to be increased by a factor of $1.3R_p$.

Per section 1621.3.12.2 of the code, the only permitted expansion anchors for non-vibration-isolated equipment over 10 hp are undercut anchors.

Static Analysis as Defined in 1997 Uniform Building Code

The 1997 UBC specifies a design lateral force F_p for nonstructural components as

$$F_p = \left(a_p C_a \frac{I_p}{R_p}\right)\left(1 + \frac{3h_x}{h_r}\right) W_p \tag{5}$$

but F_p need not be greater than

$$F_p = 4C_a I_p W_p \tag{6}$$

nor less than

$$F_p = 0.7C_a I_p W_p \tag{7}$$

where

a_p = component amplification factor in accordance with Table 6

C_a = seismic coefficient (determined from Figure 2 and Tables 7, 8, 9, and 10 as appropriate)

R_p = component response modification factor in accordance with Table 6

I_p = component importance factor in accordance with Table 11

$1 + \frac{3h_x}{h_r}$ = height amplification factor where h_x is height of attachment in structure and h_r is average height of roof above grade. The value of $h_x \geq 0$ and h_x/h_r need not exceed 1.

W_p = weight of equipment, including all items attached to or contained in equipment

Static Analysis as Defined in 1994 Uniform Building Code

(*Note*: This version of the Code is obsolete in most jurisdictions. Prior to use, its applicability must be verified by local authorities.)

The total design lateral seismic force is given as

$$F_p = ZI_pC_pW_p \tag{8}$$

where

F_p = total design lateral seismic force

Z = seismic zone factor

I_p = importance factor (set equal to 1.5 for equipment)

C_p = horizontal force factor

W_p = weight of equipment

Figure 2 and Table 10 may be used to determine the seismic zone. The seismic zone factor Z can then be determined from Table 7.

Table 4 Values of Site Coefficient F_a as Function of Site Class and Mapped Spectral Response Acceleration at 1 s Period (S_s)

Site Class	Soil Profile Name	Mapped Spectral Response Acceleration at Short Periods[a] $S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	Hard rock	0.8	0.8	0.8	0.8	0.8
B	Rock	1.0	1.0	1.0	1.0	1.0
C	Very dense soil and soft rock	1.2	1.2	1.1	1.0	1.0
D[c]	**Stiff soil profile**	**1.6**	**1.4**	**1.2**	**1.1**	**1.0**
E	Soft soil profile	2.5	1.7	1.2	0.9	b
F	See 2000 IBC Table 1615.1.1 and Note b					

[a]Use straight-line interpolation for intermediate values of mapped spectral acceleration at short period S_s.
[b]Site-specific geotechnical investigation and dynamic site response analyses must be performed to determine appropriate values.
[c]D is the default Site Class unless otherwise stated in the approved geotechnical report.

Table 5 Seismic Groups and Building Importance Factors (2000 IBC)

Seismic Group	Building Importance Factor	Nature of Occupancy
I	1.0	Buildings and other structures except those listed in Groups II, III, IV
II	1.25	1. Structures where more than 300 people congregate in one area
		2. Structures with elementary, secondary school or day care facilities with capacity of over 250
		3. Structures with capacity greater than 500 for colleges or adult education facilities
		4. Health care facilities with capacity of 50 or more resident patients but no surgery or emergency treatment facilities
		5. Jails and detention facilities
		6. Any other occupancy with occupant load greater than 5000
		7. Power-generating stations, water treatment for potable water, wastewater treatment facilities and other public utility facilities not included in Group III
		8. Buildings and other structures not included in Group III containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released
III	1.5	1. Hospitals and other health care facilities with surgery or emergency treatment facilities
		2. Fire, rescue, and police stations and emergency vehicle garages
		3. Designated earthquake, hurricane, or other emergency shelters
		4. Designated emergency preparedness, communication and operation centers and other facilities required for emergency response
		5. Power-generating stations and other public utility facilities required as emergency back-up facilities for Group III structures
		6. Structures containing highly toxic materials as defined by Section 307 where quantity of material exceeds the exempt amounts of Table 307.7(2)
		7. Aviation control towers, air traffic control centers and emergency aircraft hangers
		8. Structures with critical national defense functions
		9. Water treatment facilities required to maintain water pressure for fire suppression
IV	1	Structures that represent a low threat to human life in the event of failure including, but not limited to agricultural facilities, certain temporary facilities, and minor storage facilities

Source: International Building Code (2000). Copyright © 2000, International Code Council, Inc., Falls Church, VA. *2000 International Building Code*. Reprinted with permission of the author. All rights reserved.

Table 6 Coefficients for Mechanical Components (1997 UBC)

Mechanical and Electrical Component or Element	a_p	R_p
Equipment		
Tanks and vessels	1.0	3.0
Electrical, mechanical, plumbing equipment and piping, ducts and conduit	1.0	3.0
Equipment braced or anchored below center of mass	2.5	3.0
Emergency power, communication equipment and batteries/fuel for equipment use		
Other		
Rigid components with ductile material and attachments	1.0	3.0
Rigid components with nonductile material and attachments	1.0	1.5
Flexible components with ductile material and attachments	2.5	3.0
Flexible components with nonductile material and attachments	2.5	1.5
Vibration isolated		
Nonshallow or expansion anchor attachment	2.5	1.5
Shallow or expansion anchor attachment	2.5	0.75

Source: Uniform Building Code (1997). Reproduced from the 1997 edition of the *Uniform Building Code*™, Copyright © 1997, with permission of the publisher, the International Conference of Building Officials. ICBO assumes no responsibility for the accuracy or the completeness of summaries provided herein.

Table 7 Seismic Zone Factor *Z* (1994 and 1997 UBC)

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

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Seismic Design for Buildings (Army, Navy, and Air Force 1992) can also be used to determine the seismic zone.

The importance factor I_p from the UBC (ICBO 1994) ranges from 1 to 1.5, depending on the building occupancy and hazard level. For equipment, I_p should be conservatively set at 1.5.

The horizontal force factor C_p is determined from Table 12 based on type of equipment, the tie-down configuration, and type of base.

The weight W_p of equipment should include all items attached or contained in the equipment.

APPLYING STATIC ANALYSIS

The forces acting on a piece of equipment are vertical and lateral forces resulting from the earthquake, the force of gravity, and forces at the restraints that hold the equipment in place. The analysis assumes that the equipment does not move during the earthquake and that the relative accelerations between its center of gravity and the ground generate forces that must be balanced by reactions at the restraints. Guidance from the code bodies indicates that equipment can be analyzed as though it were a rigid component; however, a factor a_p is applied within the computation to address flexibility issues on particular equipment types or flexible mounting arrangements. (*Note*: for dynamic analysis, it is common to use a 5% damping factor for equipment and a 1% damping factor for piping.) Although the basic force computation is different, the details of load distribution in the examples that follow apply independently of the code used.

Table 8 Seismic Coefficient C_a (1997 UBC)

Soil Profile Type	Seismic Zone Factor, *Z*				
	1	2A	2B	3	4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	Requires site-specific dynamic response analysis				

Source: Uniform Building Code (1997). Reproduced from the 1997 edition of the *Uniform Building Code*™, Copyright © 1997, with permission of the publisher, the International Conference of Building Officials. ICBO assumes no responsibility for the accuracy or the completeness of summaries provided herein.

General soil profile descriptions are as follows [see UBC (1997) for more detailed information]:

S_A = Hard rock >5000 ft/s shear wave velocity
S_B = Rock with shear wave velocity > 2500 ft/s but < 5000 ft/s
S_C = Dense soil/soft rock with shear wave velocity > 1200 ft/s but < 2500 ft/s
S_D = Stiff soil with shear wave velocity > 600 ft/s but < 1200 ft/s
S_E = Soil with shear wave velocity < 600 ft/s
S_F = Soils requiring site-specific evaluation

Table 9 Near Source Factor N_a (1997 UBC)

Seismic Source Type	Closest Distance to Known Seismic Source		
	≤1.25 miles	3.1 miles	≥6.2 miles
A	1.5	1.2	1.0
B	1.3	1.0	1.0
C	1.0	1.0	1.0

Source: Uniform Building Code (1997). Reproduced from the 1997 edition of the *Uniform Building Code*™, Copyright © 1997, with permission of the publisher, the International Conference of Building Officials. ICBO assumes no responsibility for the accuracy or the completeness of summaries provided herein.

Notes: 1. Interpolation between tabulated values is permitted.
2. Source type must be as determined by approved geological data.

The forces acting on the restraints include both shear and tensile components. The application direction of the lateral seismic acceleration can vary and is unknown. Depending on its direction, it is likely that not all of the restraints will be affected or share the load equally. It is important to determine the worst-case combination of forces at all restraint points for any possible direction that the lateral wave front can follow to ensure that the attachment is adequate.

Once the overall seismic forces F_p and F_{pv} have been determined (as indicated in the previous section or per the local code requirement), the loads at the restraint points can be determined. There are many different valid methods that can be used to determine these loads, but this section suggests a couple of simple approaches.

Under some instances (particularly those relating to life support issues in hospital settings), newer code requirements indicate that critical equipment must be seismically qualified to ensure its continued operation during and after a seismic event. Special care must be taken in these situations to ensure that equipment has been shaker tested or otherwise certified to meet the maximum anticipated seismic load.

COMPUTATION OF LOADS

The forces acting on the equipment are the lateral and vertical forces resulting from the earthquake, the force of gravity, and the forces of the restraint holding the equipment in place. The analysis assumes the equipment does not move during an earthquake; thus, the sum of the forces and moments must be zero. When calculating the overturning moment, including an uplift factor, the vertical component F_{pv} at the center of gravity is typically defined to be

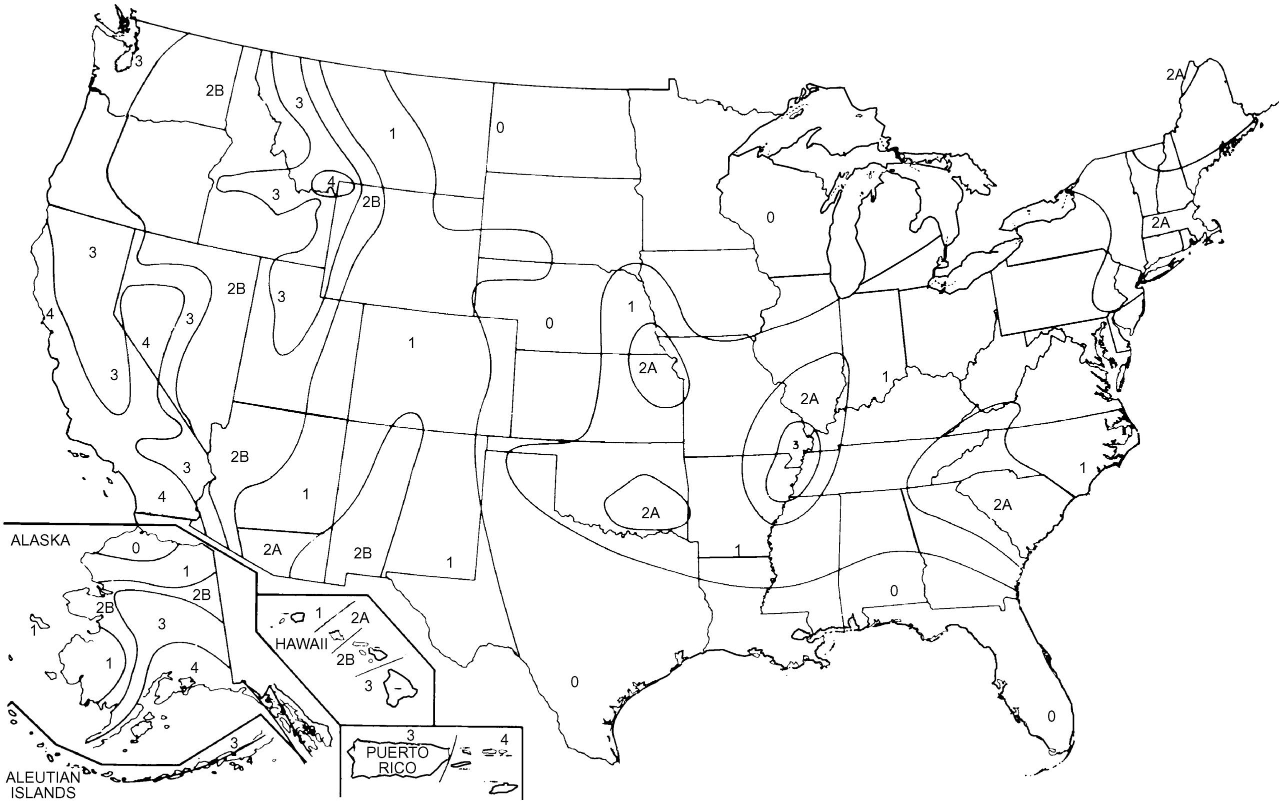

Fig. 2 Seismic Zone Map of the United States (1994 and 1997 UBC)

$$F_{pv} = F_p/3 \text{ (for older codes)} \tag{9a}$$

$$F_{pv} = 0.5C_a I_p W_p \text{ (for the 1997 UBC if specified)} \tag{9b}$$

$$F_{pv} = 0.2S_{DS} D \text{ (for the 2000 IBC)} \tag{9c}$$

when using allowable strength-based design factors, where

D = dead load affect
S_{DS} = short-period spectral response (0.2 s)

The forces of the restraint holding the equipment in position include shear and tensile forces. It is important to determine the number of bolts that are affected by the earthquake forces. The direction of the lateral force should be evaluated in both horizontal directions, as shown in Figure 3. All bolts or as few as a single bolt may be affected.

Simple Case

Figure 3 shows a rigid floor-mount installation of a piece of equipment with the center of gravity at the approximate center of the restraint pattern. To calculate the shear force, the sum of the forces in the horizontal plane is

$$0 = F_p - V \tag{10}$$

The effective shear force per restraint V_{eff} is

$$V_{eff} = F_p/N_{bolt} \tag{11}$$

where N_{bolt} is the number of bolts in shear.

The equipment shown in Figure 3 has two bolts on each side, so that four bolts are in shear. Using a single-axis moment equation to calculate the tension force, the sum of the moments for overturning are as follows:

$$F_p h_{cg} - (W_p - F_p/3)(D_1/2) - TD_1 = 0$$

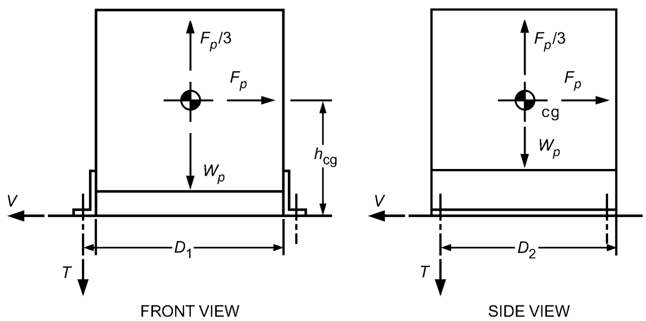

Fig. 3 Equipment with Rigidly Mounted Structural Bases

Thus,

$$T = [F_p h_{cg} - (W_p - F_p/3)(D/2)]/D \quad \text{(where } D = D_1 \text{ or } D_2\text{)} \tag{12}$$

For the example shown in Figure 3, two bolts are in tension. The effective tension force T_{eff}, where overturning affects only one side, is

$$T_{eff} = T/N_{bolt} \tag{13}$$

Shear and tension forces V and T should be calculated independently for both axes, as shown in the front and side views. The results must be combined using the larger result + 0.3 times the smaller result to account for out-of-axis forces.

Equations (9), (10), and (11) may be applied to ceiling-mounted equipment. Equation (12) must be modified to Equation (14) because the mass of the equipment adds to the overturning moment. Summing the moments determines the effective tension force on one axis as

Table 10 International Seismic Zones (1994 and 1997 UBC)

Country	City	Seismic Zone
Albania	Tirana	3
Algeria	Algiers	4
	Oran	4
Angola	Luanda	0
Antigua and Barbuda	St. Johns	3
Argentina	Buenos Aires	0
Armenia	Yerevan	3
Australia	Brisbane	1
	Canberra	1
	Melbourne	1
	Perth	1
	Sydney	1
Austria	Salzburg	2B
	Vienna	2B
Azerbaijan	Baku	3
Bahamas	Nassau	0
Bahrain	Manama	0
Bangladesh	Dhaka	3
Barbados	Bridgetown	3
Belarus	Minsk	1
Belgium	Antwerp	1
	Brussels	1
Belize	Belize City	1
Benin	Cotonou	0
Bermuda	Hamilton	0
Bolivia	La Paz	3
Botswana	Gaborone	0
Brazil	Belo Horizonte	0
	Brasilia	0
	Porto Alegre	0
	Recife	0
	Rio de Janeiro	0
	São Paulo	1
Brunei	Bandar Seri Begawan	1
Bulgaria	Sofia	3
Burkina Faso	Ouagadougou	0
Burma	Mandalay	3
	Rangoon	2B
Burundi	Bujumbura	3
Cameroon	Douala	0
	Yaounde	0
Canada	Calgary	1
	Halifax	1
	Montreal	2A
	Ottawa	2A
	Quebec	3
	Toronto	1
	Vancouver	3
Cape Verde	Praia	0
Central African Republic	Bangui	0
Chad Republic	N'Djamena	0
Chile	Santiago	4
China	Beijing (Peking)	3
	Chengdu	3
	Guangzhou (Canton)	2B
	Shanghai	2B
	Shenyang (Mukden)	4
Colombia	Barranquilla	2B
	Bogota	3
Congo	Brazzaville	0
Costa Rica	San Jose	3
Cuba	Havana	1
Cyprus	Nicosia	3
Czech Republic	Prague	1
Denmark	Copenhagen	1
Djibouti	Djibouti	3
Dominican Republic	Santo Domingo	3
Ecuador	Guayaquil	3
	Quito	4
Egypt	Alexandria	2B
	Cairo	2B
El Salvador	San Salvador	4
Equatorial Guinea	Malabo	0
Estonia	Tallinn	2A
Ethiopia	Addis Ababa	3
	Asmara	3
Fiji Islands	Suva	3
Finland	Helsinki	1
France	Bordeaux	2B
	Lyon	1
	Marseille	2B
	Paris	0
	Strasbourg	2B
Gabon	Libreville	0
Gambia	Banjul	0
Georgia	Tbilisi	3
Germany	Berlin	0
	Bonn	1
Germany (*Continued*)	Dusseldorf	1
	Frankfurt	1
	Hamburg	0
	Munich	1
	Stuttgart	2B
Ghana	Accra	3
Greece	Athens	3
	Thessaloniki	4
Grenada	St. George's	3
Guatemala	Guatemala City	4
Guinea	Conakry	0
Guinea-Bissau	Bissau	0
Guyana	Georgetown	0
Haiti	Port-au-Prince	3
Honduras	Tegucigalpa	3
Hong Kong	Hong Kong	2B
Hungary	Budapest	2B
Iceland	Reykjavik	4
India	Bombay	3
	Calcutta	2B
	Madras	1
	New Delhi	2B
Indonesia	Jakarta	3
	Medan	3
	Surabaya	3
Iraq	Baghdad	2B
Ireland	Dublin	0
Israel	Jerusalem	3
	Tel Aviv	1
Italy	Florence	3
	Genoa	2B
	Milan	2B
	Naples	2B
	Palermo	4
	Rome	2B
Ivory Coast	Abidjan	3
Jamaica	Kingston	3
Japan	Fukuoka	3
	Kobe	3
	Naha	3
	Okinawa	3
	Osaka	3
	Sapporo	3
	Tokyo	4
Jordan	Amman	3
Kazakhstan	Alma-Ata	4
Kenya	Nairobi	2B
Korea	Seoul	2A
Kuwait	Kuwait	1
Kyrgyzstan	Bishkek	4
Laos	Vientiane	1
Latvia	Riga	1
Lebanon	Beirut	3
Lesotho	Maseru	2B
Liberia	Monrovia	1
Lithuania	Vilnius	1
Luxembourg	Luxembourg	1
Madagascar	Antananarivo	0
Malawi	Lilongwe	3
Malaysia	Kuala Lumpur	1
Mali Republic	Bamako	0
Malta	Valletta	2B
Martinique	Martinique	3
Mauritania	Nouakchott	0
Mauritius	Port Louis	0
Mexico	Cuidad Juarez	2B
	Guadalajara	3
	Hermosillo	3
	Matamoros	0
	Merida	0
	Mexico City	3
	Monterrey	0
	Nuevo Laredo	0
	Tijuana	3
Moldova	Kishinev	2B
Morocco	Casablanca	2B
	Rabat	2B
Mozambique	Maputo	2B
Nepal	Kathmandu	3
Netherlands	Amsterdam	0
	The Hague	0
Netherlands Antilles	Curacao	3
New Zealand	Auckland	2B
	Wellington	4
Nicaragua	Managua	4
Niger Republic	Niamey	0
Nigeria	Kaduna	0
	Lagos	0
Norway	Oslo	2B
Oman	Muscat	2B
Pakistan	Islamabad	4
	Karachi	2B
	Lahore	2B
	Peshawar	3
Panama	Panama City	2B
Papua New Guinea	Port Moresby	3
Paraguay	Asuncion	0
Peru	Lima	4
Philippines	Baguio	3
	Cebu	4
	Manila	4
Poland	Krakow	2B
	Poznan	1
	Warsaw	1
Portugal	Azor	3
	Lisbon	3
	Oporto	2B
	Ponta Delgada	3
Qatar	Doha	0
Romania	Bucharest	3
Russia	Khabarovsk	1
	Moscow	1
	St. Petersburg	0
	Vladivostok	1
Rwanda	Kigali	3
Saudi Arabia	Dhahran	0
	Jeddah	2B
	Riyadh	0
Senegal Republic	Dakar	0
Seychelles Islands	Victoria	0
Sierra Leone	Freetown	0
Singapore	Singapore	1
Slovakia	Bratislava	2B
Somalia	Mogadishu	0
South Africa	Cape Town	2B
	Durban	2B
	Johannesburg	2B
	Pretoria	2B
Spain	Barcelona	2B
	Bilbao	2B
	Madrid	0
Sri Lanka	Colombo	0
Sudan	Khartoum	2B
Suriname	Paramaribo	0
Swaziland	Mbabane	2B
Sweden	Stockholm	0
Switzerland	Bern	2B
	Geneva	1
	Zurich	2B
Syrian Arab Republic	Damascus	3
Taiwan	Taipei	4
Tajikistan	Dushanbe	4
Tanzania	Dar Es Salaam	2B
	Zanzibar	2B
Thailand	Bangkok	1
	Chiang Mai	2B
	Songkhla	0
Togo	Lome	1
Trinidad and Tobago	Port of Spain	3
Tunisia	Tunis	3
Turkey	Adana	2B
	Ankara	2B
	Istanbul	4
	Izmir	4
Turkmenistan	Ashkhabad	4
Uganda	Kampala	2B
Ukraine	Kiev	1
United Arab Emirates	Abu Dhabi	0
	Dubai	0
United Kingdom	Belfast	0
	Edinburgh	1
	London	1
Uruguay	Montevideo	0
Uzbekistan	Tashkent	3
Vatican City	Vatican City	2B
Venezuela	Caracas	3
	Maracaibo	2B
Vietnam	Ho Chi Minh City	0
Yemen Arab Republic	Aden City	3
	Sana	3
Yugoslavia	Belgrade	2A
	Zagreb	3
Zaire	Kinshasa	0
	Lubumbashi	2B
Zambia	Lusaka	2B
Zimbabwe	Harare	2B

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Table 11 Occupancy Category and Importance Factor (1997 UBC)

Occupancy Category	Importance Factor	Nature of Occupancy
1	1.5	Essential facilities 1. Hospitals and other health care facilities with surgery or emergency treatment facilities 2. Fire, rescue, and police stations and emergency vehicle garages 3. Structures and shelters for emergency preparedness centers 4. Structures and equipment in government communication centers and other facilities required for emergency response 5. Standby power-generating equipment for Category 1 facilities 6. Aviation control towers and emergency aircraft hangers 7. Tanks or other structures containing housing or supporting water or other fire suppression material or equipment required for the protection of Category 1, 2, or 3 structures
2	1.5	Hazardous facilities 1. Group H, Division 1, 2, 6, and 7 occupancies and structures housing or supporting toxic or explosive chemicals or substances 2. Nonbuilding structures housing, supporting, or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2, or 7 occupancy
3	1.0	Special occupancy structures 1. Group A, Division 1, 2, and 2.1 occupancies 2. Buildings housing Group E, Division 1 and 3 occupancies with capacity of greater than 300 students 3. Buildings housing Group B occupancies used for college or adult education with capacity of greater than 500 students 4. Group 1, Division 1 and 2 occupancies with 50 or more resident incapacitated patients, but not included in Category 1 5. Group 1, Division 3 occupancies 6. All structures with occupancy greater than 5000 persons 7. Structures and equipment in power-generating stations and other public utility facilities not included in Category 1 or Category 2 above, and required for continued operation
4	1.0	Standard occupancy structures All structures housing occupancies or with functions not listed in Category 1, 2, or 3, and Group U occupancy towers
5	1.0	Miscellaneous structures Group U occupancy except for towers

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Table 12 Horizontal Force Factor C_p (1994 UBC)

Equipment or nonstructural components	1
Mechanical equipment, plumbing, and electrical equipment and associated piping rigidly mounted	0.75
All equipment resiliently mounted (maximum 2.0)	2

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Note: Stacks and tanks should be evaluated for compliance with applicable codes by a qualified engineer.

IBC 2000/2003 does not use seismic zones. Table 13 is a brief listing of S_s factors that can be used in IBC 2000 and 2003 to calculate the magnitude of the horizontal static seismic force acting at the equipment center of gravity. Values for IBC 2006 are available on the USGS Web site.

$$T = [F_p\, h_{cg} + (W_p + F_p/3)(D_1/2)]/D_1 \qquad (14)$$

Interaction Formula. To evaluate the combined effective tension and shear forces that act simultaneously on the bolt, use the following equation:

$$(T_{eff}/T_{allow})^{5/3} + (V_{eff}/V_{allow})^{5/3} \le 1.0 \qquad (15)$$

Allowable forces T_{allow} and V_{allow} are generic capacities given in Table 14 for wedge anchor bolts.

General Case

The classic method used to distribute seismic loads equally distributes lateral loads among the restraints and then modifies these loads as a function of the weight eccentricity. Worst-case weight, vertical seismic load, and overturning components are combined to determine a maximum vertical load component. Although simple to perform, this method tends to concentrate the seismic load resisting forces into the corners and can (particularly in seismically active areas) generate a solution that is difficult, in practice, to implement. This **polar method** is in common use and works well for most applications.

A second, somewhat more complex technique, the **lump mass method**, proportions the restraint loads based on the equipment weight and distribution. When working with larger seismic forces or unstable equipment, this offers the option of more evenly distributing the seismic load, reducing anchor size and peak restraint requirements. Because eccentric center of gravity (cg) loads are not required to be carried out to the corner restraints as in the polar method, this technique deemphasizes stresses in the equipment frame and is more suitable for nonrigid equipment types.

Note: *Although only two methods of computing forces for more general equipment cases are illustrated here, there are many other valid methods that can be used to distribute the restraint forces. It is important that any method used include the ability to account for equipment weight, seismic uplift forces, overturning forces, and an offset center of gravity within the equipment.*

Polar Method

Determination of Lateral Seismic Forces at Restraints. Lateral forces are equally distributed among the restraints. If the equipment's center of gravity does not coincide with its geometric center, a rotational factor is added to account for the imbalance. This factor is determined in three steps. First, compute the true chord length in the horizontal plane between the equipment's center of gravity and the restraints' geometric center. Second, multiply the equipment total seismic lateral force by this length (to obtain a rotational moment). Third, divide this figure by the number of moment-resisting restraints times their distance from the geometric center. (The moment-resisting restraints are those farthest and equally spaced from the geometric center.) The resulting load can then be added to

Table 13A S_s Numbers* for Selected U.S. Locations (U.S. COE 1998)

State, City	ZIP	S_s
Alabama		
Birmingham	35217	0.328
Mobile	36610	0.124
Montgomery	36104	0.170
Arkansas		
Little Rock	72205	0.461
Arizona		
Phoenix	85034	0.226
Tuscon	85739	0.325
California		
Fresno	93706	0.592
Los Angeles	90026	1.50
Oakland	94621	1.55
Sacramento	95823	0.568
San Diego	92101	1.54
San Francisco	94114	1.50
San Jose	95139	2.05
Colorado		
Colorado Springs	80913	0.178
Denver	80239	0.187
Connecticut		
Bridgeport	06606	0.332
Hartford	06120	0.274
New Haven	06511	0.285
Waterbury	06702	0.287
Florida		
Ft. Lauderdale	33328	0.070
Jacksonville	32222	0.142
Miami	33133	0.061
St. Petersburg	33709	0.078
Tampa	33635	0.083
Georgia		
Atlanta	30314	0.258
Augusta	30904	0.419
Columbia	31907	0.169
Savannah	31404	0.402
Iowa		
Council Bluffs	41011	0.186
Davenport	52803	0.130
Des Moines	50310	0.073
Idaho		
Boise	83705	0.344
Pocatello	83201	0.553
Illinois		
Chicago	60620	0.190
Moline	61265	0.135
Peoria	61605	0.174
Rock Island	61201	0.131
Rockford	61108	0.170
Springfield	62703	0.263
Indiana		
Evansville	47712	0.754
Ft. Wayne	46835	0.162
Gary	46402	0.173
Indianapolis	46260	0.182
South Bend	46637	0.121
Kansas		
Kansas City	66103	0.122
Topeka	66614	0.184
Wichita	67217	0.142
Kentucky		
Ashland	41101	0.221
Covington	41011	0.186
Louisville	40202	0.247
Louisiana		
Baton Rouge	70807	0.144
New Orleans	70116	0.130
Shreveport	71106	0.165
Massachusetts		
Boston	02127	0.325
Lawrence	01843	0.376
Lowell	01851	0.355
New Bedford	02740	0.261
Springfield	01107	0.260
Worchester	01602	0.271
Maryland		
Baltimore	21218	0.199
Maine		
Augusta	04330	0.318
Portland	04101	0.369
Michigan		
Detroit	48207	0.123
Flint	48506	0.091
Grand Rapids	49503	0.087
Kalamazoo	49001	0.116
Lansing	48910	0.109
Minnesota		
Duluth	55803	0.056
Minneapolis	55422	0.057
Rochester	55901	0.055
St. Paul	55111	0.056
Missouri		
Carthage	64836	0.149
Columbia	65202	0.178
Jefferson City	65109	0.207
Joplin	64801	0.138
Kansas City	64108	0.122
Springfield	65801	0.120
St. Joseph	64501	0.120
St. Louis	63166	0.586
Mississippi		
Jackson	39211	0.191
Montana		
Billings	59101	0.134
Butte	59701	0.599
Great Falls	59404	0.248
Nebraska		
Lincoln	68502	0.177
Omaha	68144	0.127
Nevada		
Las Vegas	89106	0.637
Reno	89509	1.29
New York		
Albany	12205	0.275
Binghampton	13903	0.185
Buffalo	14222	0.319
Elmira	14905	0.173
New York	10014	0.425
Niagara Falls	14303	0.311
Rochester	14619	0.248
Schenectady	12304	0.278
Syracuse	13219	0.192
Utica	13501	0.250
North Carolina		
Charlotte	28216	0.345
Greensboro	27410	0.255
Raleigh	27610	0.211
Winston-Salem	27106	0.281
North Dakota		
Fargo	58103	0.073
Grand Forks	58201	0.054
Ohio		
Akron	44312	0.179
Canton	44702	0.316
Cincinnati	45245	0.191
Cleveland	44130	0.197
Columbus	43217	0.164
Dayton	45440	0.206
Springfield	45502	0.216
Toledo	43608	0.171
Youngstown	44515	0.163
Oklahoma		
Oklahoma City	73145	0.339
Tulsa	74120	0.160
Oregon		
Portland	97222	1.04
Salem	97301	0.929
Pennsylvania		
Allentown	18104	0.289
Bethlehem	18015	0.304
Erie	16511	0.164
Harrisburg	17111	0.224
Philadelphia	19125	0.326
Pittsburgh	15235	0.129
Reading	19610	0.293
Scranton	18504	0.232
Rhode Island		
Providence	02907	0.267
South Carolina		
Charleston	29406	1.56
Columbia	29203	0.578
South Dakota		
Rapid City	57703	0.153
Sioux Falls	57104	0.113
Tennessee		
Chattanooga	37415	0.500
Knoxville	37920	0.589
Memphis	38109	1.25
Nashville	37211	0.305
Texas		
Amarillo	79111	0.166
Austin	78703	0.088
Beaumont	77705	0.116
Corpus Christi	78418	0.093
Dallas	75233	0.117
El Paso	79932	0.358
Ft. Worth	76119	0.110
Houston	77044	0.107
Lubbock	79424	0.099
San Antonio	78235	0.133
Waco	76704	0.095
Utah		
Salt Lake City	84111	1.79
Virginia		
Norfolk	23504	0.132
Richmond	23233	0.300
Roanoke	24017	0.290
Vermont		
Burlington	05401	0.446
Washington		
Seattle	98108	1.51
Spokane	99201	0.315
Tacoma	98402	1.23
Washington, D.C.		
Washington	20002	0.178
Wisconsin		
Green Bay	54302	0.066
Kenosha	53140	0.133
Madison	53714	0.114
Milwaukee	53221	0.120
Racine	53402	0.124
Superior	54880	0.055
West Virginia		
Charleston	25303	0.206
Huntington	25704	0.221
Wyoming		
Casper	82601	0.341
Cheyenne	82001	0.183

*Nominal values based on ZIP codes. See www.usgs.gov for calculator to check actual S_s using latitude and longitude for best results.

Table 14 Typical Allowable Loads for Wedge Anchors
(obsolete January 2007)

Diameter, in.	T_{allow}, lb	V_{allow}, lb
0.5	600	1200
0.625	900	2200
0.75	1350	3000

Notes:
1. The allowable tensile forces are for installations without special inspection (torque test) and may be doubled if the installation is inspected.
2. Additional tension and shear values may be obtained from published ICC Evaluation Service Reports.

the original (balanced) figure. This method transfers all imbalance loads to the corner restraints and provides a valid method of restraint as long as the equipment acts as a rigid body. The assumption that a piece of equipment can transfer these loads out to the corners becomes less accurate as the equipment becomes less rigid.

Figure 3 shows a typical rigid floor-mounted installation of a piece of equipment with a center of gravity coincident with the geometric center. The maximum weight on a restraint can easily be calculated using

$$W_n = -W_p/N_{bolt} + T_e \quad (16)$$

where T_e is a weight eccentricity factor computed as noted in the previous paragraph and the effective shear force V_{eff} is

$$V_{eff} = F_p/W_p(W_n) \quad (17)$$

(*Note*: For an example of a case with the center of gravity offset, see Example 3.)

Determination of Vertical Seismic Forces at Restraints. Calculation of the tensile/compressive forces at the restraints is more complex than that for determining the shear loads, and must include weight, vertical seismic force, overturning forces, and (if isolated) the type of isolator/restraint system used. The total tensile and compressive forces are the worst-case summation of each of these components. For clarity, each component is addressed here as a separate entity.

Table 13B S_s Numbers for Selected International Locations (U.S. COE 1998)

Country	City	S_s
Africa		
Algeria	Alger	1.24
	Oran	1.24
Angola	Luanda	0.06
Benin	Colonou	0.06
Botswana	Gaborone	0.06
Burkina Faso	Ougadougou	0.06
Burundi	Bujumbura	1.24
Cameroon	Douala	0.06
	Yaounde	0.06
Cape Verde	Praia	0.06
Central African Republic		
	Bangui	0.06
Chad	Ndjamena	0.06
Congo	Brazaville	0.06
Djibouti	Djibouti	1.24
Egypt	Alexandria	0.62
	Cairo	0.62
	Port Said	0.62
Equatorial Guinea		
	Malabo	0.06
Ethiopia	Addis Ababa	1.24
	Asmara	1.24
Gabon	Libreville	0.06
Gambia	Banjul	0.06
Ghana	Accra	1.24
Guinea	Bissau	0.31
	Conakry	0.06
Ivory Coast	Abidijan	0.06
Kenya	Nairobi	0.62
Lesotho	Maseru	0.62
Liberia	Monrovia	0.31
Libya	Tripoli	0.62
	Wheelus AFB	0.62
Madagascar	Tananarive	0.06
Malawi	Blantyre	1.24
	Lilongwe	1.24
	Zomba	1.24
Mali	Bamako	0.06
Mauritania	Nouakchott	0.06
Mauritius	Port Louis	0.06
Morocco	Casablanca	0.62
	Port Lyautey	0.31
	Rabat	0.62
	Tangier	1.24
Mozambique	Maputo	0.62
Niger	Niamey	0.06
Nigeria	Ibadan	0.06
	Kaduna	0.06
	Lagos	0.06
Rwanda	Kigali	1.24
Senegal	Dakar	0.06
Seychelles	Victoria	0.06
Sierra Leone	Freetown	0.06
Somalia	Mogadishu	0.06
South Africa	Cape Town	1.24
	Durban	0.62
	Johannesburg	0.62
	Natal	0.31
	Pretoria	0.62
Swaziland	Mbabane	0.62
Tanzania	Dar es Salaam	0.62
	Zanzibar	0.62
Togo	Lome	0.31
Tunisia	Tunis	1.24
Uganda	Kampaia	0.62
Zaire	Bukavu	1.24
	Kinshasa	0.06
	Lubumbashi	0.62
Zambia	Lusaka	0.62
Zimbabwe	Harare (Sallsbury)	1.24
Asia		
Afghanistan	Kabul	1.65
Bahrain	Manama	0.06
Bangladesh	Dacca	1.24
Brunei	Bandar Seri Begawan	0.31
China	Canton	0.62
	Chengdu	1.24
	Hong Kong	0.62
	Nanking	0.62
	Peking	1.65
	Shanghai	0.62
	Shengyang	1.65
	Tibwa	1.65
	Tsingtao	1.24
	Wuhan	0.62
Cyprus	Nicosia	1.24
India	Bombay	1.24
	Calcutta	0.62
	Madras	0.31
	New Delhi	1.24
Indonesia	Bandung	1.65
	Jakarta	1.65
	Medan	1.24
	Surabaya	1.65
Iran	Isfahan	1.24
	Shiraz	1.24
	Tabriz	1.65
	Tehran	1.65
Iraq	Baghdad	1.24
	Basra	0.31
Israel	Haifa	1.24
	Jerusalem	1.24
	Tel Aviv	1.24
Japan	Fukuoka	1.24
	Itazuke AFB	1.24
	Misawa AFB	1.24
	Naha, Okinawa	1.65
	Osaka/Kobe	1.65
	Sapporo	1.24
	Tokyo	1.65
	Wakkanai	1.24
	Yokohama	1.65
	Yokota	1.65
Jordan	Amman	1.24
Korea	Kwangju	0.31
	Kimhae	0.31
	Pusan	0.31
	Seoul	0.06
Kuwait	Kuwait	0.31
Laos	Vientiane	0.31
Lebanon	Beirut	1.24
Malaysia	Kuala Lumpur	0.31
Myanmar	Mandalay	1.24
	Rangoon	1.24
Nepal	Kathmandu	1.65
Oman	Muscat	0.62
Pakistan	Islamabad	1.68
	Karachi	1.65
	Lahore	0.62
	Peshawar	1.65
Qatar	Doha	0.06
Saudi Arabia	Al Badi	0.31
	Dhahran	0.31
	Jiddah	0.62
	Khamis Mushayt	0.31
	Riyadh	0.06
Singapore	All	0.31
South Yemen	Aden City	1.24
Sri Lanka	Colombo	0.06
Syria	Aleppo	1.24
	Damascus	1.24
Taiwan	All	1.65
Thailand	Bangkok	0.31
	Chinmg Mai	0.62
	Songkhia	0.06
	Udom	0.31
Turkey	Adana	0.62
	Ankara	0.62
	Istanbul	1.65
	Izmir	1.65
	Karamursel	1.24
United Arab Emirates		
	Abu Dhabi	0.06
	Dubai	0.06
Viet Nam		
	Ho Chi Minh City (Saigon)	0.06
Yemen	Sanaa	1.24
Atlantic Ocean Area		
Azorea	All	0.62
Bermuda	All	0.31
Caribbean Sea		
Bahama Islands	All	0.31
Cuba	All	0.62
Dominican Republic		
	Santo Domingo	1.24
French West Indies		
	Martinique	1.24
Grenada	Saint Georges	1.24
Haiti	Port au Prince	1.24
Jamaica	Kingston	1.24
Leeward Islands	All	1.24
Puerto Rico	All	0.83
Trinidad & Tobago		
	All	1.24
Central America		
Belize	Belmopan	0.62
Canal Zone	All	0.62
Costa Rica	San Jose	1.24
El Salvador	San Salvador	1.65
Guatemala	Guatemala	1.65
Honduras	Tegucigalpa	1.24
Mexico	Ciudad Juarez	0.62
	Guadalajara	1.24
	Hermosillo	1.24
	Matamoros	0.06
	Mazatlan	0.60
	Merida	0.06
	Mexico City	1.24
	Monterrey	0.06
	Nuevo Laredo	0.06
	Tijuana	1.24
Nicaragua	Managua	1.65
Panama	Colon	1.24
	Galeta	0.83
Europe		
Albania	Tirana	1.24
Austria	Salzburg	0.62
	Vienna	0.62
Belgium	Antwerp	0.31
	Brussels	0.62
Bulgaria	Sofia	1.24
Croatia	Zagreb	1.24
Czech Republic	Bratislava	0.62
	Prague	0.31
Denmark	Copenhagen	0.31
Finland	Helsinki	0.31
France	Bordeaux	0.62
	Lyon	0.31
	Marseille	1.24
	Nice	1.24
	Strasbourg	0.62
Germany	Berlin	0.06
	Bonn	0.62
	Bremen	0.06
	Dusseldorf	0.31
	Frankfurt	0.62
	Hamburg	0.06
	Munich	0.31
	Stuttgart	0.62
	Vaihingen	0.62
Greece	Athens	1.24
	Kavalla	1.65
	Makri	1.65
	Rhodes	1.24
	Sauda Bay	1.65
	Thessaloniki	1.65
Hungary	Budapest	0.62
Iceland	Keflavik	1.24
	Reykjavik	1.65
Ireland	Dublin	0.06
Italy	Aviano AFB	1.24
	Brindisi	0.06
	Florence	1.24
	Genoa	1.24
	Milan	0.62
	Naples	1.24
	Palermo	1.24
	Rome	0.62
	Sicily	1.24
	Trieste	1.24
	Turin	0.62
Luxembourg	Luxembourg	0.31
Malta	Valletta	0.62
Netherlands	All	0.06
Norway	Oslo	0.62
Poland	Krakow	0.62
	Poznan	0.31
	Waraszawa	0.31
Portugal	Lisbon	1.65
	Oporto	1.24
Romania	Bucharest	1.24
Russia	Moscow	0.06
	St. Petersburg	0.06
Serbia	Belgrade	0.62
Spain	Barcelona	0.62
	Bilbao	0.62
	Madrid	0.06
	Rota	0.62
	Seville	0.62
Sweden	Goteborg	0.62
	Stockholm	0.31
Switzerland	Bern	0.62
	Geneva	0.31
	Zurich	0.62
Ukraine	Kiev	0.06
United Kingdom	Belfast	0.06
	Edinburgh	0.31
	Glasgow/Renfrew	0.31
	Hamilton	0.31
	Liverpool	0.31
	London	0.62
	Londonderry	0.31
	Thurso	0.31
North America		
Greenland	All	0.31
Canada	Argentia NAS	0.62
	Calgary, AB	0.31
	Churchill, MB	0.06
	Cold Lake, AB	0.31
	Edmonton, AB	0.31
	E. Harmon, AFB	0.62
	Fort Williams, ON	0.06
	Frobisher, NT	0.06
	Goose Airport	0.31
	Halifax, NS	0.31
	Montreal, QC	1.24
	Ottawa, ON	0.62
	St. John's, NL	1.24
	Toronto, ON	0.31
	Vancouver, BC	1.24
	Winnipeg, MB	0.31
South America		
Argentina	Buenos Aires	0.25
Brazil	Belem	0.06
	Belo Horizonte	0.06
	Brasilia	0.06
	Manaus	0.06
	Porto Alegre	0.06
	Recife	0.06
	Rio de Janeiro	0.06
	Salvador	0.06
	Sao Paulo	0.31
Bolivia	La Paz	1.24
	Santa Cruz	0.31
Chile	Santiago	1.65
	Valparaiso	1.65
Colombia	Bogota	1.24
Ecuador	Quito	1.65
	Guayaquil	1.24
Paraquay	Asuncion	0.06
Peru	Lima	1.65
	Piura	1.65
Uruguay	Montevideo	0.06
Venezuela	Maracaibo	0.62
	Caracas	1.65
Pacific Ocean Area		
Australia	Brisbane	0.31
	Canberra	0.31
	Melbourne	0.31
	Perth	0.31
	Sydney	0.31
Caroline Islands	Koror, Palau	0.62
	Ponape	0.06
Fiji	Suva	1.24
Johnson Island	All	0.31
Mariana Islands	Guam	1.24
	Saipan	1.24
	Tinian	1,24
Marshall Islands	All	0.31
New Zealand	Auckland	1.24
	Wellington	1.65
Papau New Guinea		
	Port Moresby	1.24
Phillipine Islands	Cebu	1.65
	Manila	1.65
	Bagulo	1.24
Samoa	All	1.24
Wake Island	All	0.06

The nominal weight component at each restraint is simply the total operating weight divided by the number of restraints. This is always directed downward when summing forces and is computed using Equation (16).

The vertical seismic force is simply the weight component at each location multiplied by the vertical seismic force factor in terms of the total F_{pv} load expressed in gs, the gravitational constant (F_{pv}/W_p, where F_{pv} is the vertical seismic load component as defined by the code and W_p is the total operating weight of the equipment). This can be directed either upward or downward when summing forces. The actual value of this load at a given restraint point is defined by Equations (9a), (9b), and (9c). Assuming the simple case, $F_{pv} = F_p/3$, and results in a nominal load at the restraint of

$$T_n = \pm W_n F_{pv}/W_p \tag{18}$$

where T_n and W_n are tensile and gravity loads at particular restraint points.

The overturning force at a restraint is highly dependent on the direction of the seismic wave front. In the polar method, first determine a worst-case load angle and load. This is then summed with the other worst-case loads to determine a worst-case combination load.

The worst-case approach angle for the seismic wave front is a function of the length and width of the restraint. If we assume a dimension of A between the outermost restraints on the long axis and a dimension B between the outermost restraints on the short axis, then the worst-case angle for the seismic load relative to the long axis centerline is

$$\theta = \arctan(B/A) \tag{19}$$

The stabilizing factor that results from multiple restraints aligned along the long axis is

$$I_x = 2(\text{No. of restraints on } y\text{-axis side})(B/2)^2 \tag{20}$$

and along the short axis is

$$I_y = 2(\text{No. of restraints on } x\text{-axis side})(A/2)^2 \tag{21}$$

From Figure 3, F_p (total lateral seismic force), and h_{cg} (height of equipment center of gravity above point of restraint), the maximum force generated at a corner restraint by overturning is

$$T_m = \pm\left\{F_p\left[\left(\frac{Bh_{cg}}{I_x}\right)\cos\theta + \left(\frac{Ah_{cg}}{I_y}\right)\sin\theta\right]\right\} \tag{22}$$

This can be directed either upward or downward to produce a worst case when summing forces.

$$T_{eff} = -W_n + T_n + T_m \tag{23}$$

Lump Mass Method

In the lump mass method, the total equipment weight is distributed among the restraints in a manner that reflects the equipment's actual weight distribution. There are many methods of determining the distribution analytically or by testing, although they are not addressed in this section. Frequently, a weight distribution can be obtained from the equipment manufacturer.

Determination of Lateral Seismic Forces at Restraints. Once the static point loads are obtained or computed for each restraint location, they can be multiplied by the lateral seismic acceleration factor (F_p/W_p) to determine lateral forces at each restraint point. Thus, if the weight at each restraint point is W_n, then

$$V_{eff} = (F_p/W_p)W_n \tag{24}$$

Determination of Vertical Seismic Forces at Restraints. This method considers the loads at all the restraints individually and computes the overturning forces for each in 1° increments for a full 360° of possible seismic wave front angle; it is only practical to perform using a spreadsheet. The total lateral seismic force F_p is divided into x- and y-axis components for each possible wave front approach angle. These forces are multiplied by the height of the equipment center of gravity above the point of restraint h_{cg}. The resulting moments are then resolved into forces at each restraint based on the x- and y-axis moment arms associated with the particular restraint location and the proportion of the load that it will bear. For example, with four total restraints (one at each corner), two per side, each restraint will generate half the uplift force generated in resisting an x- or y-axis moment. If there are additional restraints at the equipment midpoint (six restraints total and the resistant force is proportional to the moment arm), for long-axis moments, each corner restraint will contribute 4/10 of the resisting moment and each center restraint will contribute 1/10 of the resisting moment. The x- and y-axis values are then summed to determine the total resulting force for the restraint at each load angle increment. Once forces for all angles from 0 to 360° have been computed, the worst-case load at each point can be identified.

Equations (25) through (29) are used to compute the overturning load for the case illustrated in Figure 3. They should be repeated for each restraint and for each seismic wave front angle θ from 0 to 360° with the maximum and minimum final values selected.

$$F_{px} = F_p(-\cos\phi) \quad F_{py} = F_p(-\sin\phi) \tag{25}$$

$$T_{mx} = F_{px}h_{cg}/(2D_1) \quad T_{my} = F_{py}h_{cg}/(2D_2) \tag{26}$$

where D_1 and D_2 are the spacing between restraints on the long and short axis, respectively.

$$T_m = T_{mx} + T_{my} \tag{27}$$

Because the total vertical seismic force is the total weight multiplied by the vertical seismic force factor in g, the vertical seismic force at any particular restraint is

$$T_n = \pm W_n F_{pv}/W_p \tag{28}$$

and the total vertical force component is

$$T_{eff} = -W_n + T_m + T_n \tag{29}$$

Positive (+) loads that result from these equations are identified as upward. Negative (–) loads are directed downward.

Resilient Support Factors

If the equipment being restrained is isolated, the following three factors must be considered:

- For all forces that are not directed along the principal axes, only the corner restraints can be considered to be effective. Thus, for either distribution method, only the corner restraints can be considered capable of absorbing vertical loads.
- If the restraints are independent (separate entities) from the spring isolation elements and if, when exposed to uplift loads, vertical spring forces are not absorbed within the housing of an integral isolator/restraint assembly, the weight factor determined in the first step of the vertical load analysis should be ignored. (This is because any effect that a weight reduction has on the attachment hardware forces is replaced by an approximately equal vertical force component from the spring.)
- If the 2000 IBC is used, the final computed forces must be doubled.

Interaction of Attachment Hardware Shear and Tensile Forces. The two most common attachment arrangements are directly bolting the equipment or restraint or anchoring it to concrete using post-installed (not cast in place) anchors. To evaluate the combined effective tensile and shear forces that act simultaneously on these connections, separate analyses are required.

Note that for short-term wind or seismic loads, the codes or fastener evaluation report may permit the allowable stresses for either bolted-to-steel or anchored-to-concrete connections to be increased by a factor of 1.33 (*California applications involving hospitals and schools have special requirements and should be reviewed by a California registered engineer*). If allowable stress design (ASD) data are used to size hardware for bolt-to-steel connections for 1997 UBC or 2000 IBC code applications, the loads may be reduced by a factor of 1.4. If load and resistance factor (LRF) criteria is used for the hardware, the 1.4 factor does not apply. The allowable stresses to be used when evaluating anchor bolts are drawn from ICBO test reports and reflect values that are considerably derated from values typically published by anchor manufacturers. Also note that the values published by ICBO for concrete anchors are working-stress based and in that form are incompatible with the 1997 UBC and 2000 IBC, which generate strength-based loads. Before computing interaction forces, the computed loads for these newer codes must be reduced by a factor of 1.4 to make them compatible. The 1.4 factor does not apply when working with the older codes.

For direct attachment to a 1994 UBC or older steel structure with through bolts, the design capacity of the attachment hardware should be based on criteria established in the American Institute of Steel Construction (AISC) manual. Based on the use of A307 bolts, the basic formula for computing allowable tensile stress when shear stresses are present is

$$T_{allow} = 26{,}000 - 1.8S_v \qquad (30)$$

where S_v is the shear stress in the bolt in psi. T_{allow}, the maximum allowable tensile stress, must not exceed 20,000 psi.

However, because these stresses are appropriate for dead- plus live-load combinations, they can be appropriately inflated by 1.33 when allowable stress design provisions are used and when they are used to resist wind and seismic loads as well. Peak bolt loads are based on the maximum permitted stress multiplied by the nominal bolt area.

For attachment to concrete using post-installed anchors, the 1997 UBC requires that seismic forces be doubled for isolated systems. This additional factor has been included in Table 6. On the other hand, because published post-installed anchor capacities are working-stress based, the strength-based forces predicted by the 2000 IBC or the 1997 UBC can be reduced by a factor of 1.4 when evaluating the integrity of the connection.

The interaction formula for concrete anchors is

$$(T_{eff}/T_{allow})^{5/3} + (V_{eff}/V_{allow})^{5/3} \le 1 \qquad (31)$$

where

T_{eff} = tensile load at anchor
T_{allow} = maximum allowable anchor tensile load
V_{eff} = shear load at anchor
V_{allow} = maximum allowable anchor shear load

LAG SCREWS INTO TIMBER

Acceptable loads for lag screws into timber can be obtained from the *National Design Specification*® (NDS®) *for Wood Construction* (AWC 1997). Selected fasteners must be secured to solid lumber, not to plywood or other similar material. Withdrawal force design values are a function of the screw size, penetration depth, and wood density and can be increased by a factor of 1.6 for short-term seismic or wind loads. Table 9.2A in the NDS identifies withdrawal forces on a force/imbedment depth basis. Note that the values published in the table are working-stress based and in that form are incompatible with the 1997 UBC and 2000 IBC, which generate strength-based loads. Before computing interaction forces, the computed loads for these newer codes must be reduced by a factor of 1.4 to make them compatible. The 1.4 factor does not apply when working with the older codes. In addition, NDS Table 9.4.2 introduces deration factors for reduced edge distance and bolt spacing.

In timber construction, the interaction formula given in Equation (15) does not apply. Instead, per Section 9.3.5 of the NDS, the equation is

$$Z_a' = (W'p)Z'/[(W'p)\cos^2\alpha + Z'\sin^2\alpha]$$

where

Z' = shear capacity drawn from Table 9.3A
W' = side grain withdrawal force = $1800G^{3/2}D^{3/4}$
G = specific gravity of the timber
D = diameter
p = embedment depth of screw
α = angle of composite force measured flat with surface of timber

ANCHOR BOLTS

Several types of anchor bolts for insertion in concrete are manufactured. Wedge and undercut anchors perform better than self-drilling, sleeve, or drop-in types. **Adhesive anchors** are stronger than other anchors, but lose their strength at elevated temperatures (e.g., on rooftops and in areas damaged by fire).

Wedge anchors have a wedge on the end with a small clip around the wedge. After a hole is drilled, the bolt is inserted and the external nut tightened. The wedge expands the small clip, which bites into the concrete.

When **undercut anchors** are tightened, the tip expands to seat against a shoulder cut in the bottom of the anchor hole. Although these have the highest capacity of commonly available anchor types, the cost of the extra operation to cut the shoulder in the hole greatly limits the frequency of their use in the field.

A **self-drilling anchor** is basically a hollow drill bit. The anchor is used to drill the hole and is then removed. A wedge is then inserted on the end of the anchor, and the assembly is drilled back into place; the drill twists the assembly fully in place. The self-drilling anchor is weaker than other types because it forms a rough hole.

Drop-in expansion anchors are hollow cylinders with a tapered end. After they are inserted in a hole, a small rod is driven through the hollow portion, expanding the tapered end. These anchors are only for shallow installations because they have no reserve expansion capacity.

A **sleeve anchor** is a bolt covered by a threaded, thin-wall, split tube. As the bolt is tightened, the thin wall expands. Additional load tends to further expand the thin wall. The bolt must be properly preloaded or friction force will not develop the required holding force. These anchors are typically not used in seismic applications because of the limited reserve capacity.

Adhesive anchors may be in glass capsules or installed with various tools. Pure epoxy, polyester, or vinyl ester resin adhesives are used with a threaded rod supplied by the contractor or the adhesive manufacturer. Some adhesives have a problem with shrinkage; others are degraded by heat. However, some adhesives have been tested without protection to 1100°F before they fail (all mechanical anchors will fail at this temperature). Where required, or if there is a concern, anchors should be protected with fire retardants similar to those applied to steel decks in high-rise buildings.

The manufacturer's instructions for installing the anchor bolts should be followed. Performance test data published by manufacturers should include shock, fatigue, and seismic resistance. ICBO reports have further information on allowable forces for design. Use a safety factor of 2 if the installation has not been inspected by a qualified firm or individual.

WELD CAPACITIES

Weld capacities may be calculated to determine the size of welds needed to attach equipment to a steel plate or to evaluate raised support legs and attachments. A static analysis provides the effective tension and shear forces. The capacity of a weld is given per unit length of weld based on the shear strength of the weld material. For steel welds, the allowable shear strength capacity is 16,000 psi on the throat section of the weld. The section length is 0.707 times the specified weld size.

For a 1/16 in. weld, the length of shear in the weld is 0.707 × 1/16 = 0.0442 in. The allowable weld force $(F_w)_{allow}$ for a 1/16 in. weld is

$$(F_w)_{allow} = 0.0442 \times 16{,}000 = 700 \text{ lb per inch of weld}$$

For a 1/8 in. weld, the capacity is 1400 lb/in.

The effective weld force is the sum of the vectors calculated in Equations (11) and (13), (17) and (23), or (24) and (29). Because the vectors are perpendicular, they are added by the method of the square root of the sum of the squares (SRSS), or

$$(F_w)_{allow} = \sqrt{(T_{eff})^2 + (V_{eff})^2}$$

The length of weld required is given by the following equation:

$$\text{Length} = (F_w)_{eff}/(F_w)_{allow} \tag{32}$$

SEISMIC SNUBBERS

Several types of snubbers are manufactured or field fabricated. All snubber assemblies should meet the following minimum requirements to avoid imparting excessive accelerations to HVAC&R equipment:

- Impact surface should have a high-quality elastomeric surface that is not cemented in place.
- Resilient material should be easy to inspect for damage and be replaceable.
- Snubber system must provide restraint in all directions.
- Snubber capacity should be verified either through test or by analysis and should be certified by an independent, registered engineer to avoid serious design flaws.

Typical snubbers are classified as Types A through J (Figure 4).

Type A. Snubber built into a resilient mounting. All-directional, molded bridge-bearing quality neoprene element is a minimum of 1/8 in. thick.

Type B. Isolator/restraint. Stable isolation spring bears on the base plate of the fixed restraining member. Earthquake motion of isolated equipment is restrained close to the base plate, minimizing pullout force to the base plate anchorage.

Type C. Spring isolator with built-in all-directional restraints. Restraints have molded neoprene elements with a minimum thickness of 1/8 in. A neoprene sound pad should be installed between the spring and base plate. Sound pads below the base plate are not recommended for seismic installations.

Type D. Integral all-directional snubber/restrained spring isolator with neoprene element.

Type E. Fully bonded neoprene mount capable of withstanding seismic loads in all directions with no metal-to-metal contact.

Type F. All-directional three-axis snubber with neoprene element. Neoprene element of bridge-bearing quality is a minimum of 3/16 in. thick. Snubber must have a minimum of two anchor bolt holes.

Type G. All-directional lateral snubber. Neoprene element is a minimum of 1/4 in. thick. Upper bracket is welded to the equipment.

Type H. Restraint for floor-mounted equipment consisting of interlocking steel assemblies lined with resilient elastomer. Bolted to equipment and anchored to structure through slotted holes to allow field adjustment. After final adjustment, weld anchor to floor bracket and weld angle clip to equipment or, alternatively, fill slots with adhesive grout to prevent slip.

Type I. Single-axis, single-direction lateral snubber. Neoprene element is a minimum of 1/4 in. thick. Minimum floor mounting is with two anchor bolts. Must be used in sets of four or more.

Type J. Prestretched aircraft wire rope with galvanized end connections that avoid bending the wire rope across sharp edges. This type of snubber is mainly used with suspended pipe, ducts, and equipment.

EXAMPLES

The following examples are provided to assist in the design of equipment anchorage to resist seismic forces. For Examples 1 through 4, assume 2000 IBC applies, I_p = 1.5, S_s = 0.85, site soil class is C, and the equipment is located at the top of a 50 ft building. Also include an uplift force component $F_{pv} = 0.2S_{DS}D$ where D is the dead load for all examples. Examples 1 through 5 are solved using the polar method of analysis while Example 6 is solved by the lump mass method.

Example 1. Anchorage design for equipment rigidly mounted to the structure (see Figure 5).

From Equations (1) through (3) and (9), calculate the lateral seismic force and its vertical component. Note that for expansion anchors, R_p = 1.5; for other cases, R_p = 2.5. For attachment to wood or steel, the seismic loads are initially determined using the 2.5 factor. For attachment to concrete with post-installed anchors, the 1.5 factor is used and an embedment depth of over 8 bolt diameters is assumed. For rigidly mounted mechanical equipment, a_p from Table 3 is 1.0.

The first step in the load determination process is to determine S_{DS} using the following equation and F_a = 1.1 (from Table 4, site class C):

$$S_{DS} = 2F_aS_s/3 = 2 \times 1.1 \times 0.85/3 = 0.623$$

Using this value for S_{DS} Equation (1) gives

$$F_p = [(0.4 \times 1.0 \times 0.623 \times 1000)/(2.5/1.5)]\,(1 + 2 \times 50/50) = 450 \text{ lb}$$
(steel/wood anchorage)

$$F_p = [(0.4 \times 1.0 \times 0.623 \times 1000)/(1.5/1.5)]\,(1 + 2 \times 50/50) = 750 \text{ lb}$$
(concrete anchorage)

Equation (2) shows that F_p need not be greater than

$$1.6 \times 0.85 \times 1.5 \times 1000 = 2040 \text{ lb}$$

Equation (3) shows that F_p must not be less than

$$0.3 \times 0.85 \times 1.5 \times 1000 = 383 \text{ lb}$$

Therefore F_p = 450 lb for attachment to steel or wood and 750 lb for attachment to concrete.

$$F_{pv} = 0.2S_{DS}D = 0.2 \times 0.623 \times 1000 = 125 \text{ lb}$$

For through-bolt or lag screw attachment, the attachment forces are as follows:

Calculate the overturning moment OTM:

$$\text{OTM} = F_p h_{cg} = (450 \times 40) = 18{,}000 \text{ in}\cdot\text{lb} \tag{33}$$

Calculate the resisting moment RM:

$$\text{RM} = (W_p \pm F_{pv})d_{min}/2 = (1000 \pm 125)28/2 = 15{,}750 \text{ or } 12{,}250 \text{ in}\cdot\text{lb} \tag{34}$$

Calculate the tension force T, using RM_{min} to determine the maximum tension force:

$$T = (\text{OTM} - \text{RM}_{min})/d_{min} = (18{,}000 - 12{,}250)/28 = 205 \text{ lb} \tag{35}$$

This force is the same as that obtained using Equation (12).

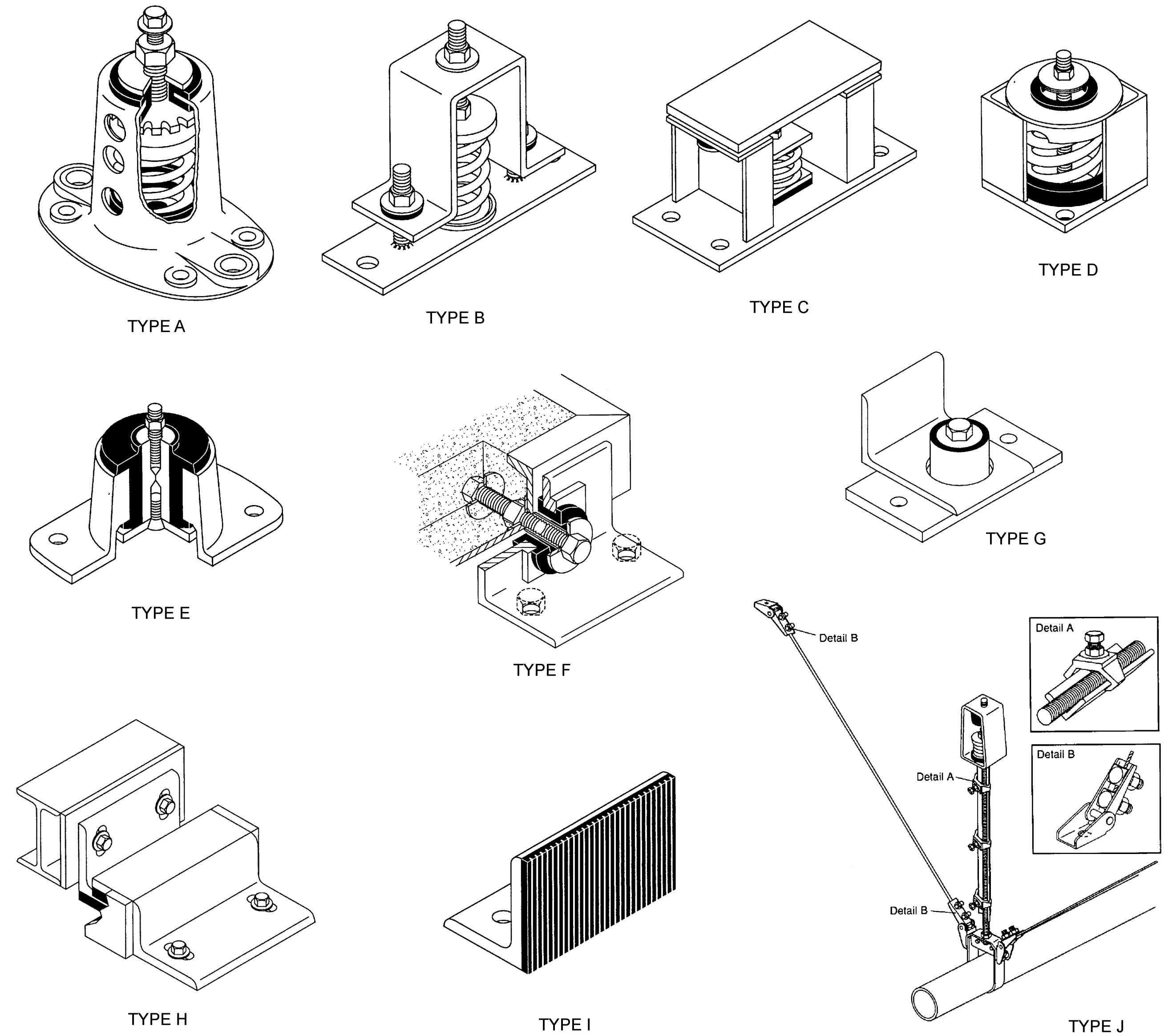


Fig. 4 Seismic Snubbers

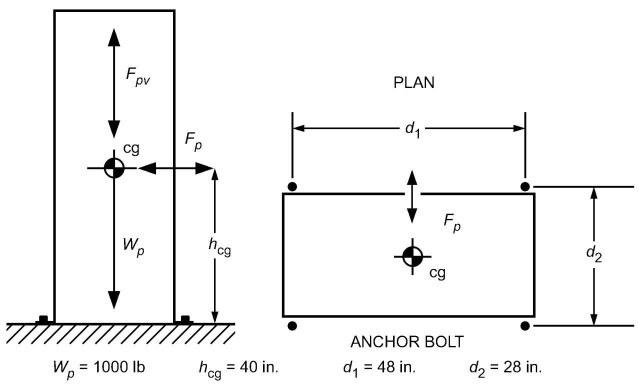


Fig. 5 Equipment Rigidly Mounted to Structure (Example 1)

Calculate T_{eff} per bolt from Equation (13):

$$T_{eff} = 205/2 = 103 \text{ lb per through-bolt or lag screw}$$

Calculate shear force per bolt from Equation (11):

$$V_{eff} = 450/4 = 112.5 \text{ lb per through-bolt or lag screw}$$

For post-installed anchor bolts, the attachment forces are computed in the same manner, using F_p = 750 lb instead of 450 lb:

The resulting T_{eff} = 317 lb and V_{eff} = 187.5 lb (post-installed anchor bolt)

(*Note*: Per 1621.3.12.2 of the 2000 IBC, if over 10 hp, the use of an undercut anchor bolt is required.)

Step Case 1. *Equipment attached to a timber structure*

Before computing interaction forces, the computed loads must be reduced by a factor of 1.4 to make them compatible with the capacity data listed in the *National Design Specification*® (*NDS*®) *for Wood Construction* (AWC 1997). The lateral load V_{eff} becomes 112.5/1.4 or 80.4 lb per bolt and the pullout load T_{eff} becomes 103/1.4 = 73.5 lb per bolt. For the capacity of the connection, a resulting combined load and angle relative to the mounting surface must be computed. The combined load is

$$T\alpha_{eff} = \sqrt{V_{eff}^2 + T_{eff}^2} = \sqrt{80.4^2 + 73.5^2} = 109 \text{ lb}$$

The angle $\alpha = \arcsin(T_{eff}/Z'_\alpha) = 42.5°$, where Z' is the allowable lag screw load multiplied by applicable factors and Z'_α is the factored allowable lag screw load at angle α from the mounting surface.

Selected fasteners must be secured to solid lumber, not to plywood or other similar material. The following calculations are made to determine whether a 1/2 in. diameter, 4 in. long lag screw in redwood will hold the required load. For this computation, it is assumed that bolt spacing, edge distance, temperature, and other factors do not reduce the bolt capacity (see NDS for further details) and that the load allowable factor for short-term wind or seismic loads is 1.6.

From Table 9.3A in the NDS, for redwood, $G = 0.37$, and Z perpendicular to the grain is 512 lb.

From Table 9.2A in the NDS, for $G = 0.37$ and 3.5 in. full thread, $W = 385 \times 3.5 = 1350$ lb

Substituting into the combined load equation

$$Z'_\alpha = \frac{(W'_p)Z'}{(W'_p)\cos^2\alpha + Z'\sin^2\alpha}$$

gives

$$Z'_\alpha = \frac{(385 \times 3.5)512}{(385 \times 3.5)\cos^2 42.5 + (512)\sin^2(42.5)} = 714 \text{ lb}$$

$$T\alpha_{eff}/Z'_\alpha = 109/714 = 0.15 < 1.0$$

Therefore, a 1/2 in. diameter, 4 in. long lag screw can be used at each corner of the equipment.

Step Case 2. *Equipment attached to concrete with post-installed anchors*

As determined previously,

$$T_{eff} = 317 \text{ lb per bolt and } V_{eff} = 187.5 \text{ lb per bolt}$$

Because these loads are strength based (2000 IBC and 1997 UBC) and published ICBO anchor allowables are stress based, the loads can be reduced by a factor of 1.4 to bring the data into alignment with the computation. The net result and figures used to complete the analysis are

$$T_{eff} = 317/1.4 = 226 \text{ lb per bolt}$$

and

$$V_{eff} = 187.5/1.4 = 134 \text{ lb per bolt}$$

It is good design practice to specify a minimum of 1/2 in. diameter bolts to attach roof- or floor-mounted equipment to the structure. Determine whether 1/2 in. wedge anchors without special inspection provisions will hold the required load.

From Table 14, $T_{allow} = 600$ lb and $V_{allow} = 1200$ lb

From Equation (15),

$$(317/600)^{5/3} + (134/1200)^{5/3} = 0.37 < 1.0$$

Therefore, 1/2 in. diameter, 4 in. long post-installed anchors can be used. If special inspection of the anchor installation is provided by qualified personnel, T_{allow} only may be increased by a factor of 2.

Step Case 3. *Equipment attached to steel*

For equipment attached directly to a steel member, analysis is the same as that shown in case 1. Allowable values for the attaching bolts are given in the *Manual of Steel Construction* (AISC 1989). Values for A307 bolts are given in Table 15. Because these values are stress-based numbers, the 1.4 factor used for lag screws and post-installed anchor bolts is also applicable. If the numbers used were LRF values, the 1.4 factor would not be appropriate Also, the interaction formula in Equation (15) does not apply to steel-to-steel connections. Instead, the allowable tension load must be modified as in the following equation:

$$(T_{allow})_{mod} = F_t A_b$$

where A_b is the nominal bolt area.

In this case (with a 1/2 in. diameter bolt), from Equation (30), $V_{eff} = 187.5/1.4$, $N_{bolt} = 1$, and $A_b = 0.196$.

Table 15 Allowable Loads for A307 Bolts per AISC (1989)

Diameter, in.	T_{allow}, lb	V_{allow}, lb	A_b, in^2
1/2	3900	1950	0.196
5/8	6100	3100	0.307
3/4	8800	4400	0.442
1	15,700	7900	0.785

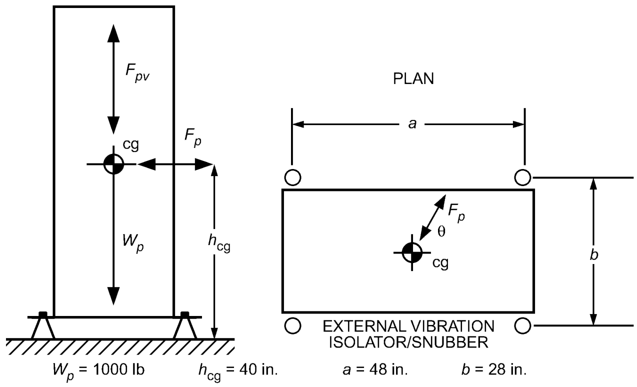


Fig. 6 Equipment Supported by External Spring Mounts

$$F_t = 26{,}000 - 1.8(V_{eff}/N_{bolt})A_b \le 20{,}000(4/3) = 26{,}670$$

V_{eff} is in pounds and F_t is in psi. The 33% stress increase (4/3) is allowed for short-term loads such as wind or earthquakes.

$$T_{allow} = 25{,}995 \times 0.196 = 5095 > 317/1.4 = 226 \text{ lb}$$

Therefore, 1/2 in. diameter bolts can be used.

Example 2. Anchorage design for equipment supported by external spring mounts (Figure 6) and attached to concrete using nonshallow post-installed anchors.

A mechanical or acoustical consultant should choose the type of isolator or snubber or combination of the two. Then the product vendor should select the actual spring snubber.

Using the 2000 IBC, the lateral force F_p must be recalculated using new factors. S_{DS} remains as in Example 1. For expansion anchors, $R_p = 1.5$, and for resiliently mounted mechanical equipment, A_p from Table 3 is 2.5.

The basic force equation is then

$$F_p = [(0.4 \times 2.5 \times 0.623 \times 1000)/(1.5/1.5)](1 + 2 \times 50/50) = 1869 \text{ lb}$$

Equation (2) indicates that F_p need not be greater than

$$1.6 \times 0.85 \times 1.5 \times 1000 = 2040 \text{ lb}$$

Equation (3) indicates that F_p must not be less than

$$0.3 \times 0.85 \times 1.5 \times 1000 = 383 \text{ lb}$$

The vertical force F_{pv} equals

$$F_{pv} = 0.2 S_{DS} D = 0.2 \times 0.623 \times 1000 = 125 \text{ lb}$$

Because the equipment is resiliently supported, section 1621.3.1 of the 2000 IBC indicates that the computed forces should be doubled. Therefore, F_p is $1869 \times 2 = 3738$ lb and F_{pv} is $125 \times 2 = 250$ lb

Assume that the center of gravity cg of the equipment coincides with the center of gravity of the isolator group.

If T = maximum tension on isolator

C = maximum compression on isolator

then

$$T = \frac{-W_p + F_{pv}}{4} + F_p h_{cg}\frac{\cos\theta}{2b} + F_p h_{cg}\frac{\sin\theta}{2a}$$

$$= \frac{-W_p + F_{pv}}{4} + \frac{F_p h_{cg}}{2}\left(\frac{\cos\theta}{b} + \frac{\sin\theta}{a}\right)$$

To find maximum T or C, set $dT/d\theta = 0$:

$$\frac{dT}{d\theta} = \frac{F_p h_{cg}}{2}\left(-\frac{\sin\theta}{b}\frac{\cos\theta}{a}\right) = 0$$

$$\theta_{max} = \tan^{-1}(b/a) = \tan^{-1}(28/48) = 30.26° \quad (36)$$

$$T = \frac{-W_p + F_{pv}}{4} + \frac{F_p h_{cg}}{2}\left(\frac{\cos\theta_{max}}{b} + \frac{\sin\theta_{max}}{a}\right) \quad (37)$$

$$C = \frac{-W_p + F_{pv}}{4} - \frac{F_p h_{cg}}{2}\left(\frac{\cos\theta_{max}}{b} + \frac{\sin\theta_{max}}{a}\right) \quad (38)$$

$$T = \frac{-1000 + 250}{4} + \frac{3738 \times 40}{2} \times \left(\frac{\cos 30.26}{28} + \frac{\sin 30.26}{48}\right) = 2908 \text{ lb}$$

$$C = \frac{-1000 + 250}{4} - \frac{3738 \times 40}{2} \times \left(\frac{\cos 30.26}{28} + \frac{\sin 30.26}{48}\right) = -3408 \text{ lb}$$

Calculate the shear force per isolator:

$$V = (F_p/N_{iso}) = 3738/4 = 935 \text{ lb} \quad (39)$$

This shear force is applied at the operating height of the isolator. Uplift tension T on the vibration isolator is the worst condition for the design of the anchor bolts. The compression force C must be evaluated to check the adequacy of the structure to resist the loads (Figure 7).

$$(T_1)_{eff} \text{ per bolt} = T/2 = 2908/2 = 1454 \text{ lb}$$

The value of $(T_2)_{eff}$ per bolt due to overturning on the isolator is

$$(T_2)_{eff} = \frac{V \times \text{Operating height}}{dN_{bolt}}$$

where d is the distance from edge of isolator base plate to center of bolt hole.

$$(T_2)_{eff} = (1454 \times 8)/(3 \times 2) = 1939 \text{ lb}$$

$$(T_{max})_{eff} = (T_1)_{eff} + (T_2)_{eff} = 1454 + 1939 = 3393 \text{ lb}$$

$$V_{eff} = 935/2 = 468 \text{ lb}$$

Determine whether 5/8 in. post-installed anchors *with special inspection* will handle this load. From Equation (15) and Table 9,

$$\left(\frac{3393}{2 \times 900}\right)^{5/3} + \left(\frac{468}{2200}\right)^{5/3} = 2.95 > 1.0$$

Therefore, 5/8 in. post-installed anchors will not carry the load.

Example 3. Anchorage design for equipment with a center of gravity different from that of the isolator group (Figure 8).

Anchor properties $\quad I_x = 4B^2; \quad I_y = 4L^2$

Angles $\quad \theta = \tan^{-1}(B/L) \quad (40)$

$$\alpha = \tan^{-1}(e_x/e_y) \quad (41)$$

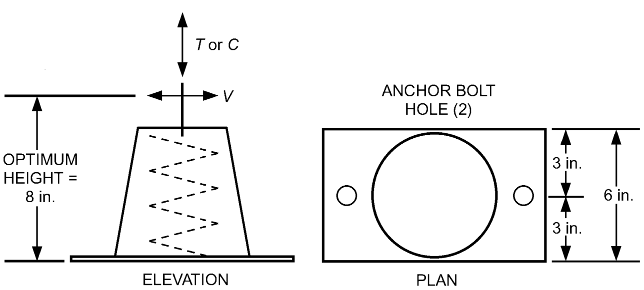


Fig. 7 Spring Mount Detail (Example 2)

$$\beta = 180\,|\alpha - \theta| \quad (42)$$

$$\phi = \tan^{-1}(LI_x/BI_y) \quad (43)$$

Vertical reactions

$$(W_n)_{max/min} = W_p \pm F_{pv} \quad (44)$$

Vertical reaction caused by overturning moment

$$T_m = \pm F_p\left(\frac{Bh_{cg}}{I_x}\cos\phi + \frac{Lh_{cg}}{I_y}\sin\phi\right) \quad (45)$$

Vertical reaction caused by eccentricity

$$(T_e)_{max/min} = (W_n)_{max/min}\left(\frac{Be_y}{I_x} + \frac{Le_x}{I_y}\right) \quad (46)$$

Vertical reaction caused by W_p

$$(T_w)_{max/min} = (W_n)_{max/min}/4 \quad (47)$$

$$T_{max} = T_m + (T_e)_{max} + (T_w)_{max} \text{ (always compression)} \quad (48)$$

$$T_{min} = -T_m + (T_e)_{min} + (T_w)_{min} \text{ (tension if negative)} \quad (49)$$

Horizontal reactions

Horizontal reaction caused by rotation

$$V_{rot} = F_p\left[\frac{e_x^2 + e_y^2}{16(B^2 + L^2)}\right]^{0.5} \quad (50)$$

$$V_{dir} = F_p/4 \quad (51)$$

$$V_{max} = (V_{rot}^2 + V_{dir}^2 - 2V_{rot}V_{dir}\cos\beta)^{0.5} \quad (52)$$

For isolated equipment attachment to steel, Equation (1) gives

$$F_p = 0.4 \times 2.5 \times 0.623 \times 2500 \times \frac{1.5}{2.5}\left(1 + 2 \times \frac{50}{50}\right) = 2804 \text{ lb}$$

Equation (2) indicates that F_p need not be greater than

$$1.6 \times 0.85 \times 1.5 \times 2500 = 5100 \text{ lb}$$

Equation (3) indicates that F_p must not be less than

$$0.3 \times 0.85 \times 1.5 \times 2500 = 956 \text{ lb}$$

The vertical force F_{pv} is

$$F_{pv} = 0.2S_{DS}D = 0.2 \times 0.623 \times 2500 = 312 \text{ lb}$$

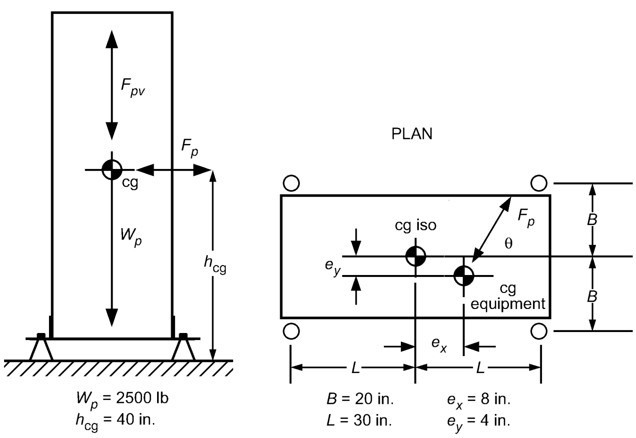


Fig. 8 Equipment with Center of Gravity Different from Isolator Group (in Plan View)

Doubling the result for the resilient supports gives a design F_p of 5608 lb and F_{pv} of 624 lb.

Anchor properties are

$$I_x = 4(20)^2 = 1600$$

$$I_y = 4(30)^2 = 3600$$

From Equations (40) through (43),

$$\theta = 33.69° \qquad \alpha = 63.43°$$

$$\beta = 150.26° \qquad \phi = 33.69°$$

From Equation (44),

$$(W_n)_{max/min} = 2500 \pm 624 = 3124 \text{ lb or } 1876 \text{ lb}$$

From Equation (45),

$$T_m = \pm 5608(0.416 + 0.184) = \pm 3365 \text{ lb}$$

From Equation (46),

$$(T_e)_{max/min} = 0.1167(W_n)_{max/min} = 365 \text{ lb or } 219 \text{ lb}$$

From Equation (47),

$$(T_w)_{max/min} = 781 \text{ lb or } 469 \text{ lb}$$

From Equation (48),

$$T_{max} = 3365 + 365 + 781 = 4511 \text{ lb}$$

From Equation (49),

$$T_{min} = -3365 + 219 + 469 = -2677 \text{ lb (tension)}$$

From Equations (50), (51), and (52),

$$V_{rot} = 5608 \times 0.062 = 348 \text{ lb}$$

$$V_{dir} = 5608/4 = 1402 \text{ lb}$$

$$V_{max} = 1713 \text{ lb}$$

The values of T_{min} and V_{max} are used to design the anchorage of the isolators and/or snubbers, and T_{max} is used to verify the structure's adequacy to resist the vertical loads.

Example 4. Anchorage design for equipment with supports and bracing for suspended equipment (Figure 9). Equipment weight W_p = 500 lb.

Because post-installed anchors may not withstand published allowable static loads when subjected to vibratory loads, vibration isolators should be used between the equipment and the structure to damp vibrations generated by the equipment.

For attachment to steel and isolated equipment, Equation (1) gives

$$F_p = 0.4 \times 2.5 \times 0.623 \times 500 \times \frac{1.5}{2.5}\left(1 + 2 \times \frac{50}{50}\right) = 561 \text{ lb}$$

Equation (2) indicates that F_p need not be greater than

$$1.6 \times 0.85 \times 1.5 \times 500 = 1020 \text{ lb}$$

Equation (3) indicates that F_p must not be less than

$$0.3 \times 0.85 \times 1.5 \times 500 = 191 \text{ lb}$$

The vertical force F_{pv} is

$$F_{pv} = 0.2 S_{DS} D = 0.2 \times 0.623 \times 500 = 62 \text{ lb}$$

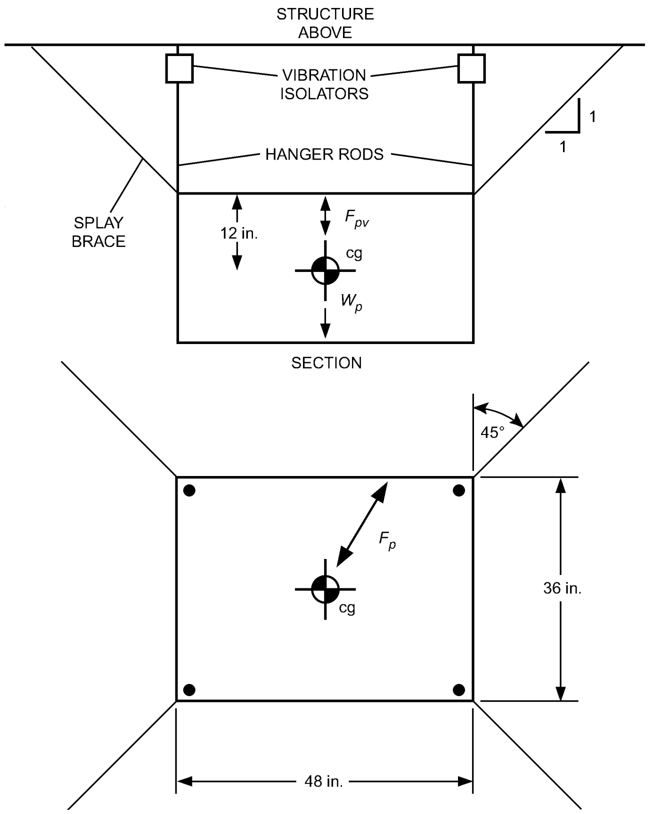
Doubling the result for the resilient supports gives a design F_p of 1122 lb and F_{pv} of 124 lb.

Anchor properties $\quad I_x = 4B^2$; $I_y = 4L^2$

Angle $\quad \phi = \tan^{-1}(LI_x/BI_y) = 36.86°$

From Equation (44),

$$(W_n)_{max/min} = 500 \pm 124 = 624 \text{ lb or } 376 \text{ lb}$$



Note: Splay braces are prestretched aircraft cables with enough slack so that isolators can fully function vertically.

Fig. 9 Supports and Bracing for Suspended Equipment

From Equation (45),

$$T_m = \pm 1122\,(0.132 + 0.075) = \pm 233 \text{ lb}$$

From Equation (46),

$$T_e = 0$$

From Equation (47),

$$(T_w)_{max/min} = 156 \text{ lb or } 94 \text{ lb}$$

From Equation (48),

$$(T_{eff})_{max} = 233 + 0 + 156 = 389 \text{ lb (downward)}$$

From Equation (49),

$$(T_{eff})_{min} = -233 + 0 + 94 = -139 \text{ lb (upward)}$$

Forces in the hanger rods:

Maximum tensile = 389 lb
Maximum compression = 139 lb

Force in the splay brace = $F_p\sqrt{2}$ = 1587 lb at a 1:1 slope

Because of the force being applied at the critical angle, as in Example 2, only one splay brace is effective in resisting the lateral load F_p.

Design of hanger rod/vibration isolator and connection to structure

When post-installed anchors are mounted to the underside of a concrete beam or slab, the allowable tension loads on the anchors must be reduced to account for cracking of the concrete. A general rule is to use half the allowable load. Some manufacturers have ICC reports that provide allowable values for anchors installed under the slab.

Determine whether a 1/2 in. wedge anchor with special inspection provisions will hold the required load.

$$T_{allow} = 600 \times 0.5 \times 2 = 600 \text{ lb} > T_{eff} = 389 \text{ lb}$$

Therefore, a 1/2 in. rod and post drill-in anchor should be used at each corner of the unit.

For anchors installed without special inspection,

$$T_{allow} = 600 \times 0.5 = 300 \text{ lb} < T_{eff} = 389 \text{ lb}$$

Therefore, a larger anchor should be chosen.

Determine if the 1/2 in. hanger rod would require a stiffener if it is 36 in. long.

Good practice is to not allow the compressive force to exceed 1/4 of the Euler buckling load. The Euler load can be computed using the formula

$$P_e = \frac{\pi^2 EI}{L^2} \tag{53}$$

where

E = modulus of rod (29,000,000 psi for steel)
I = moment of inertia ($\pi d^4/64$)
L = rod length, in.
d = rod diameter, in.

which gives

$$P_e = \pi^2(29{,}000{,}000 \text{ psi} \times 0.0031)/36^2 = 685 \text{ lb}$$

Because the maximum compressive load of 139 lb is less than 685 lb/4, a rod stiffener is not required.

Design of splay brace and connection to structure

Force in the slack cable = 1587 lb

Because all of the load must be resisted by a single cable, the forces in the connection to the structure are

$$V_{max} = 1122 \text{ lb} \qquad T_{max} = F_p = 1122 \text{ lb}$$

Determine whether a 3/4 in. wedge anchor will hold the required load.

From Table 14,

$$T_{allow} = 1350/2 = 675 \text{ lb} \qquad V_{allow} = 3000 \text{ lb}$$

From Equation (31),

$$\left(\frac{1122}{675}\right)^{5/3} + \left(\frac{1122}{3000}\right)^{5/3} = 2.52 > 1.0$$

Therefore, it is not permissible to use a 3/4 in. anchor. A larger anchor or multiple anchors bolted through a clip and to the structure are required.

Because the cable forces are relatively small, a 3/8 in. aircraft cable attached to clips with cable clamps should be used. The clips, in turn, may be attached to either the structure or the equipment.

Example 5. Anchorage design for equipment with a center of gravity different from that of the isolator group and with six mounting points (Figure 10).

For this computation, use the 2000 IBC and a ground acceleration term S_S of 1.29. Assume that the general-purpose building (I_p = 1) is built on stiff soil (F_a = 1.4). The equipment (a_p = 1) is solid-mounted 2/3 of the way to the top of the building and is attached to concrete (R_p = 1.5). The equipment weighs 1000 lb and the specification vertical load is equal to $0.2S_{DS}D$.

To determine S_{DS}, use the equation

$$S_{DS} = \frac{2F_aS_s}{3} = \frac{2 \times 1.4 \times 1.29}{3} = 1.21 \tag{54}$$

For both of these cases, use lateral force Equation (1):

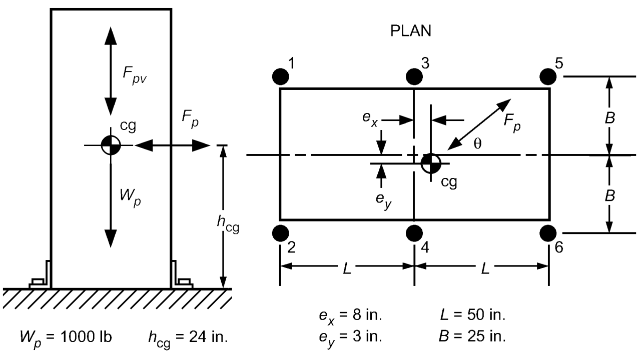


Fig. 10 Floor-Mounted Equipment with Solid or Isolated Connection to Structure (Example 5)

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

where F_p need not exceed $1.6S_{DS}I_pW_p$, nor can it be less than $0.35S_{DS}I_pWp$.

Lateral design force

Inserting appropriate values into Equation (1) gives

$$F_p = (0.4 \times 1 \times 1.21 \times W_p)(1/1.5)[1 + 2(0.66)] = 0.75\ W_p$$

where F_p need not exceed $1.6 \times 1.21 \times 1 \times W_p$ or $1.93W_p$, nor can it be less than $0.3 \times 1.21 \times 1 \times W_p$ or $0.36W_p$.

In this case,

$$F_p = 0.75\ W_p = 0.75 \times 1000 = 750 \text{ lb} \tag{55}$$

Vertical design force

The vertical seismic load is

$$F_{pv} = 0.2 \times 1.21 \times 1000 = 0.242W_p = 242 \text{ lb} \tag{56}$$

Doubling the result for the resilient supports gives a design F_p of 1500 lb and F_{pv} of 484 lb.

For calculating maximum tension and compression loads, apply F_p as shown. To add the rotational effect of the applied load to the shear calculation, apply F_p at an angle perpendicular to r_0 where r is the radius from the geometric center to the center of gravity.

Anchor properties

$$I_x = 6B^2 = 3750 \text{ in}^2 \qquad I_y = 4L^2 = 10{,}000 \text{ in}^2$$

$$r^2_{1,2,5,6} = B^2 + L^2 = 3125 \text{ in}^2 \qquad r^2_{3,4} = B^2 = 625 \text{ in}^2$$

$$r_0 = \sqrt{e_x^2 + e_y^2} = 8.54 \text{ in.} \qquad r_{max} = \sqrt{B^2 + L^2} = 55.9 \text{ in}^2$$

$$\sum r^2 = 13{,}750 \text{ in}^2$$

The equation for calculating the maximum loads at any point in this system caused by bending about a specified axis (assuming that operating clearances are minimal and all load points contribute) is

$$f_b = \frac{M_z r_{max}}{I}$$

where M_z is the moment, r_{max} is the longest r_n dimension, and I is the rotational resistance factor.

For biaxial bending about two orthogonal axes, the equation becomes

$$f_b = \frac{(M_xIy - M_yIxy)L}{IxIy - Ixy^2} + \frac{(M_xIx - M_xIxy)B}{IxIy - Ixy^2}$$

When more than four load points are effective in countering rotational forces, angle θ = angle ϕ.

From Equation (43),

$$\phi = \tan^{-1}(LI_x/BI_y) \text{ and therefore } \theta = \tan^{-1}(LI_x/BI_y)$$

$$\phi = \theta = 36.87°$$

From Equation (44),

$$(W_n)_{max/min} = 1484 \text{ lb or } 516 \text{ lb}$$

From Equation (45) and setting angle θ = angle ϕ,

$$T_{my} = \pm 300 \text{ lb}$$

From Equation (46),

$$(T_e)_{max/min} = 89 \text{ lb or } 31 \text{ lb}$$

Modifying Equation (47) for 6 restraints,

$$(T_w)_{max\,\S\,min} = \frac{(W_n)_{max\,\S\,min}}{6} = 247 \text{ lb or } 86 \text{ lb}$$

From Equation (48),

$$(T_{eff})_{max} = 300 + 89 + 247 = 636 \text{ lb (downward)}$$

From Equation (49),

$$(T_{eff})_{min} = -300 + 29 + 86 = -185 \text{ lb (upward)}$$

To calculate the maximum shear loads, add the direct shear loads to the rotational loads.

$$V_{dir} = F_p/6 = 250 \text{ lb}$$

$$M_z = F_p \times r_0 = 12{,}810 \text{ in}\cdot\text{lb}$$

$$V_{rot} = \frac{M_z r_{max}}{\sum r^2} = 52 \text{ lb}$$

$$V_{max} = V_{dir} + V_{rot} = 302 \text{ lb}$$

It is slightly conservative to add these numbers linearly.

Example 6. Anchorage design for equipment with a center of gravity different from that of the isolator group using lump mass analysis format. Use the information given in Example 5.

The computed global seismic loads are the same as in Example 5 and are computed in the same manner.

The final result, assuming resilient supports, yields a design F_p of 1500 lb and F_{pv} of 484 lb.

Details on the methods used to approximate the dead-weight loads at each restraint point, based on an off-center weight distribution, are not illustrated here. There are several valid methods to do this; these loads are often available from the equipment supplier and are given for this example as follows:

Static loads by location

Loc 1 = 80 lb	Loc 2 = 101 lb
Loc 3 = 208 lb	Loc 4 = 265 lb
Loc 5 = 152 lb	Loc 6 = 193 lb

Lateral load computation

The lateral force at each location equals the seismic coefficient ($R_p = 2 \times 0.75 = 1.5$) multiplied by the weight at each location, which gives

Lateral seismic loads by location

Loc 1 = 120 lb	Loc 2 = 152 lb
Loc 3 = 312 lb	Loc 4 = 398 lb
Loc 5 = 228 lb	Loc 6 = 290 lb

Vertical load computation

First, the weight component at each location from the equipment must be accounted for. This is the W_n factor and was previously identified.

The vertical load component F_{pv} at each point is computed by multiplying the vertically applied load expressed in gs by the weight at each location. This factor is determined from Equation (56), taking into consideration the doubling of F_{pv} due to the use of resilient supports:

$$F_{pv} = 0.484 \times 1000 = 484 \text{ lb or } 0.484\,g$$

Variation due to vertical force F_{pv} by location (W_n)

Loc 1 = ±58 lb	Loc 2 = ±74 lb
Loc 3 = ±151 lb	Loc 4 = ±193 lb
Loc 5 = ±110 lb	Loc 6 = ±140 lb

To compute the overturning forces, composite forces must be computed and summed for each restraint and each possible seismic wavefront angle from 0 to 360°. The worst cases of these loads are then identified and summed with the weight components. Because this determination is suited to the use of a spreadsheet, the entire calculation is not shown here. Instead, the forces for one restraint position and one wavefront angle are computed to illustrate the procedure.

Computation for Loc 1 and wavefront angle ϕ = 30°

The x component of the F_p force = $F_{px} = F_p(-\cos\phi)$. Therefore,

$$F_{px} = 1500(-\cos 30) = -1299 \text{ lb} \tag{57}$$

Similarly, the y component of the F_p force = $F_{py} = F_p(-\sin\phi)$, thus

$$F_{py} = 1500(-\sin 30) = -750 \text{ lb} \tag{58}$$

To compute the force on the restraint at Loc 1 due to the load F_{px}, first determine the share of the long-axis load that the end restraints in a system with six restraints will absorb.

To begin, sum the moments around the restraints at one end of the base and assume that the vertical force components at the other restraints are proportional to the distance from that center of rotation. If the vertical restraints have no operating clearance, the magnitude of the force for the pair of central restraints is 1/2 of that for the end restraints. In addition, the moment arm for the end restraint is twice that of the arm for the central restraints. As a result, the end restraints have four times the effect on the resisting moment as the central restraints. Because there are two restraints on each end, each restraint generates 40% of the moment. A similar analysis can be conducted for different arrangements of restraints. On the opposite axis (with only two restraints) each restraint takes 1/2 of the load. If there is sufficient clearance in the central vertical restraints, they will not come into play when the equipment "rocks" back and forth. In that case, the equipment should be evaluated as being supported on four points when computing overturning loads. In this example, minimum vertical clearance is assumed.

$$T_{mx} = F_{px} h_{cg} 0.4/(2L) = -1299 \times 24 \times 0.4/(2 \times 50) = -125 \text{ lb} \tag{59}$$

and

$$T_{my} = F_{py} h_{cg} 0.5/(2B) = 750 \times 24 \times 0.5/(2 \times 25) = 180 \text{ lb} \tag{60}$$

Combining these loads gives a total net overturning force at Loc 1 (with a seismic wavefront angle of 30°) equal to

$$T_m = T_{mx} + T_{my} = 125 + (-180) = -55 \text{ lb (upward)} \tag{61}$$

Totaling the worst case combination of vertical loads for this location and this angle gives us

$$T_{max1} = W_{n1} + W_{u1} + T_{m1} = 80 + 58 - 55 = 83 \text{ lb (downward)} \tag{62}$$

$$T_{min1} = W_{n1} - W_{u1} + T_{m1} = 80 - 58 - 55 = -33 \text{ lb (upward)} \tag{63}$$

A complete analysis requires that this be repeated for each restraint and each angle from 0 to 360°.

Prescriptive provisions of IBC 2000 can be summarized as follows:

- Formulas for relative displacement of floor and ceiling can be conservatively estimated at 1% of the floor-to-ceiling height. This displacement must be used to determine the required horizontal flexibility of the pipe, duct, or electrical connections at the equipment interface.

- Noncompact anchors with an embedment-to-bolt diameter less than 8 must reduce R_p from 2.5 to 1.5. This increases the seismic calculated force F_p by 1.66.
- In IBC 2003, using all-directional snubbers with clearance of more than 1/4 in. increases F_p by a factor of 2; in IBC 2000, this increase is for all isolated equipment.
- Component supports must be designed to accommodate component movement to prevent pounding on the structure or other components. This affects internal isolators and snubbers.
- Equipment components exposed to seismic impact forces and using nonductile housings must be designed using 25% of material yield stresses.
- Nonessential equipment, failure of which can cause essential equipment failure, must be designed as essential equipment.
- If the structure's site class is not provided in the contract documents, assume site class D, subject to change by the building official.
- Wedge anchor bolts are allowed with isolated equipment snubbers in IBC 2000 but not in IBC 2003 or NFPA *Code* 5000.
- For pipe or duct on any given run, if the distance from the bottom of the structure to the top of the support is 12 in. or less for all supports in that run, then that run does not need sway braces.
- Pipe and ducts may not be required to have sway braces, depending on size, material content, and importance factor. These conditions are defined in the code itself and SMACNA Appendix B (1998).

INSTALLATION PROBLEMS

The following should be considered when installing seismic restraints.

- Anchor location affects the required strengths. Concrete anchors should be located away from edges, stress joints, or existing fractures. ASTM *Standard* E488 should be followed as a guide for edge distances and center-to-center spacing.
- Concrete anchors should not be too close together. Adhesive anchors can be closer together than expansion anchors. Expansion anchors (self-drilling and drop-in) can crush the concrete where they expand and impose internal stresses in the concrete. Spacing of all anchor bolts should be carefully reviewed. (See manufacturer's recommendations.)
- Supplementary steel bases and frames, concrete bases, or equipment modifications may void some manufacturers' warranties. Snubbers, for example, should be properly attached to a subbase. Bumpers may be used with springs.
- Static analysis does not account for the effects of resonant conditions within a piece of equipment or its components. Because all equipment has different resonant frequencies during operation and nonoperation, the equipment itself might fail even if the restraints do not. Equipment mounted inside a housing should be seismically restrained to meet the same criteria as the exterior restraints.
- Snubbers used with spring mounts should withstand motion in all directions. Some snubbers are only designed for restraint in one direction; sets of snubbers or snubbers designed for multidirectional purposes should be used.
- Equipment must be strong enough to withstand the high deceleration forces developed by resilient restraints.
- Flexible connections should be provided between equipment that is braced and piping and ductwork that need not be braced.
- Flexible connections should be provided between isolated equipment and braced piping and ductwork.
- Bumpers installed to limit horizontal motion should be outfitted with resilient neoprene pads to soften the potential impact loads of the equipment.
- Anchor installations should be inspected; in many cases, damage occurs because bolts were not properly installed. To develop the rated restraint, bolts should be installed according to manufacturer's recommendations.
- Brackets in structural steel attachments should be matched to reduce bending and internal stresses at the joint.
- With the exception of heavy-duty clamps used to attach longitudinal restraints to piping systems, friction must not be relied on to resist any load. All connections should be positive and all holes should be tight-fitting or grouted to ensure minimal clearance at the attachment points.

WIND RESTRAINT DESIGN

Damage done to HVAC&R equipment by both sustained and gusting wind forces has increased concern about the adequacy of equipment protection defined in design documents. The following calculative procedure generates the same type of total design lateral force used in static analysis of the seismic restraint. The value determined for the design wind force F_w can be substituted for the total design lateral seismic force F_p when evaluating and choosing restraint devices.

The American Society of Civil Engineers' (ASCE) *Standard* 7-95 includes design guidelines for wind, snow, rain, and earthquake loads. Note that the equations, guidelines, and data presented here are from an earlier version of the ASCE standard and only cover nonstructural components. The current standard (2005) includes more comprehensive and rigorous procedures for evaluating wind forces and wind restraint. Refer to the latest version of ASCE *Standard* 7 adopted by the local jurisdiction.

TERMINOLOGY

Classification. Buildings and other structures are classified for wind load design according to Table 16.

Basic wind speed. The fastest mile-per-hour wind speed at 33 ft above the ground of Terrain Exposure C (see Table 17) having an annual probability of occurrence of 0.02. Data in ASCE *Standard* 7 or regional climatic data may be used to determine basic wind speeds. ASCE data do not include all special wind regions (such as mountainous terrains, gorges, and ocean promontories) where records or experience indicate that the wind speeds are higher than what is shown in appropriate wind data tables. For these circumstances, regional climatic data may be used provided that both acceptable extreme-value statistical analysis procedures were used in reducing the data and that due regard was given to the length of record, averaging time, anemometer height, data quality, and terrain exposure. One final exclusion is that tornadoes were not considered in developing the basic wind speed distributions.

Design wind force. Equivalent static force that is assumed to act on a component in a direction parallel to the wind and not necessarily normal to the surface area of the component. This force varies with respect to height above ground level.

Importance factor *I*. A factor that accounts for the degree of hazard to human life and damage to HVAC components (Table 18). For hurricanes, the value of the importance factor can be linearly interpolated between the ocean line and 100 miles inland because wind effects are assumed negligible at this distance inland.

Gust response factor *G*. A factor that accounts for the fluctuating nature of wind and the corresponding additional loading effects on HVAC components.

Minimum design wind load. The wind load may not be less than 10 lb/ft^2 multiplied by the area of the HVAC component projected on a vertical plane that is normal to the wind direction.

CALCULATIONS

Two procedures are used to determine the design wind load on HVAC components. The **analytical procedure**, described here, is the most common method for standard component shapes. The second method, the **wind-tunnel procedure**, is used in the analysis of complex and unusually shaped components or equipment located on sites that produce wind channeling or buffeting because of upwind obstructions. The analytical procedure produces design wind forces that are expected to act on HVAC components for durations of 1 to 10 s. The various factors, pressure, and force coefficients incorporated in this procedure are based on a mean wind speed that corresponds to the fastest mile-per-hour wind speed.

Analytical Procedure

The design wind force is determined by the following equation:

$$F_w = Q_z G C_f A_f \tag{64}$$

where

F_w = design wind force, lb
Q_z = velocity pressure evaluated at height z above ground level, lb/ft^2
G = gust response factor for HVAC components evaluated at height z above ground level
C_f = force coefficient (Table 19)
A_f = area of HVAC component projected on a plane normal to wind direction, ft^2

Certain of the preceding factors must be calculated from equations that incorporate site-specific conditions that are defined as follows:

Velocity Pressure. The design wind speed must be converted to a velocity pressure that is acting on an HVAC component at a height z above the ground. The equation is

$$Q_z = 0.00256 K_z V^2 I \tag{65}$$

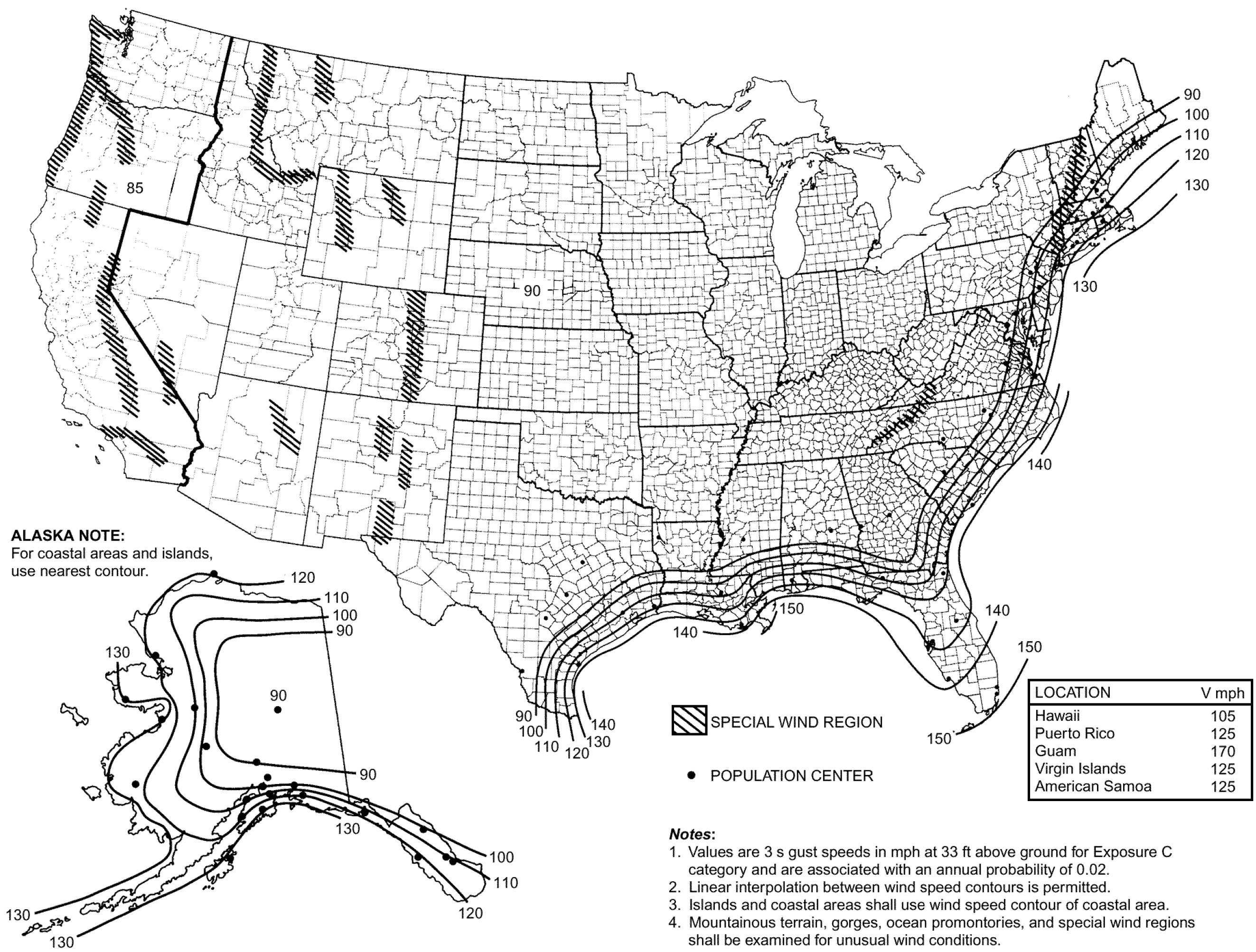
where

K_z = velocity pressure exposure coefficient from Table 20
V = velocity from Figure 11, mph
I = importance factor from Table 18

Example Calculations: Analytical Procedure

The following example calculations are for a 400 ton cooling tower with dimensions shown in Figure 12:

Tower height h = 10 ft
Tower width D = 10 ft
Tower length l = 20 ft
Tower operating weight W_p = 19,080 lb
Tower diagonal dimension = $\sqrt{10^2 + 20^2}$ = 22.4 ft
Area normal to wind direction A_f = 10 × 22.4 = 224 ft^2
From Table 19; C_f = 1.0 for wind acting along diagonal with h/D = 10/10 = 1.



LOCATION	V mph
Hawaii	105
Puerto Rico	125
Guam	170
Virgin Islands	125
American Samoa	125

Notes:
1. Values are 3 s gust speeds in mph at 33 ft above ground for Exposure C category and are associated with an annual probability of 0.02.
2. Linear interpolation between wind speed contours is permitted.
3. Islands and coastal areas shall use wind speed contour of coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Fig. 11 Wind Speed Data
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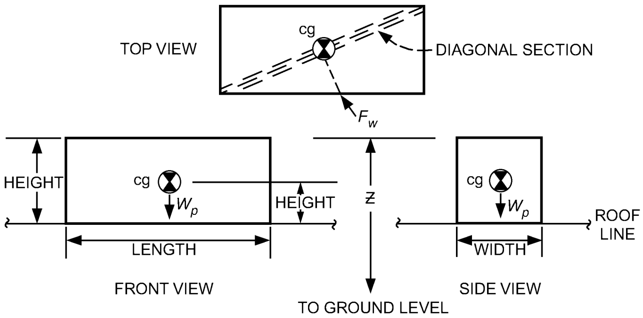

Fig. 12 Equipment Dimensions and Force Locations for Wind Examples 7, 8, and 9

Table 16 Definition of Exposure Categories

Exposure A. Large city centers with at least 50% of the buildings having a height in excess of 70 ft. Use of this exposure category is limited to those areas for which terrain representative of Exposure A prevails upwind for at least 0.5 mile or 10 times the height of the building or structure, whichever is greater.
Possible channeling effects or increased velocity pressures caused by building or structure location in the wake of adjacent buildings needs to be considered.

Exposure B. Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions the size of single-family dwellings or larger. Use of this exposure category is limited to those areas for which terrain representative of Exposure B prevails upwind for at least 1500 ft or 10 times the height of the building or structure, whichever is greater.

Exposure C. Open terrain with scattered obstructions having heights generally less than 30 ft. This category includes flat open country and grasslands.

Exposure D. Flat, unobstructed areas exposed to wind flowing over open water for a distance of at least 1 mile. This exposure applies only to those buildings and other structures exposed to the wind coming from over the water. Exposure D extends inland from the shoreline for 1500 ft or 10 times the height of the building or structure, whichever is greater.

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Notes:
1. HVAC components for buildings with mean roof height of 60 ft or less are designed on the basis of Exposure C.
2. HVAC components on buildings with mean roof height greater than 60 ft and other structures are designed on basis of exposure categories defined in this table, except assume Exposure B for buildings and other structures sited in terrain representative of Exposure A.

Table 17 Wind Importance Factor *I* (Wind Loads)

Category	*I*
I	0.87
II	1
III	1.15
IV	1.15

Table 18 Exposure Category Constants

Exposure Category	α	Z_g, ft	Gust Factor *G*
A	5	1500	0.8
B	7	1200	0.8
C	9.5	900	0.85
D	11.5	700	0.85

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Note: See Table 17 for definitions of Exposure Categories.

Table 19 Force Coefficients for HVAC Components, Tanks, and Similar Structures

		C_f for *h*/*D* Values of		
Shape	**Type of Surface**	**1**	**7**	**25**
Square (wind normal to face)	All	1.3	1.4	2.0
Square (wind along diagonal)	All	1.0	1.1	1.5
Hexagonal or octagonal ($D\sqrt{Q_z} > 2.5$)	All	1.0	1.2	1.4
Round ($D\sqrt{Q_z} > 2.5$)	Moderately smooth	0.5	0.6	0.7
	Rough ($D'/D = 0.02$)	0.7	0.8	0.9
	Very rough ($D'/D = 0.08$)	0.8	1.0	1.2
Round ($D\sqrt{Q_z}$ £ 2.5)	All	0.7	0.8	1.2

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Notes:
1. Design wind force calculated based on area of structure projected on a plane normal to the wind direction. Force is assumed to act parallel to wind direction.
2. Linear interpolation may be used for *h*/*D* values other than shown.
3. Nomenclature:
   D = diameter or least horizontal dimension, ft
   D' = depth of protruding elements such as ribs and spoilers, ft
   h = structure height, ft
   Q_z = velocity pressure evaluated at height *z* above ground level, lb/ft²

Example 7. Suburban hospital in Omaha, Nebraska. The top of the cooling tower is 100 ft above ground level.

Solution:

From Figure 11, the design wind speed is found to be 90 mph.
From Table 16, use Category IV
From Table 17, use Exposure A
From Table 18, $I = 1.15$
From Table 21, $K_z = 0.68$
From Table 20, G = 0.8
Substitution into Equation (65) yields:

$$Q_z = 0.00256 \times 0.68 \times (90)^2 \times 1.15 = 16.22 \text{ lb/ft}^2$$

Substitution into Equation (64) yields the design wind force as

$$F_w = 16.22 \times 0.8 \times 1.0 \times 224 = 2906 \text{ lb}$$

Example 8. Office building in New York City. Top of tower is 600 ft above ground level.

Solution:

From Figure 11, the design wind speed is 120 mph.
From Table 16, use Category II.
From Table 17, use Exposure A
From Table 18, $I = 1.0$
Because $z > 500$ ft, K_z must be determined from Note 2 of Table 21.
From Table 20, $\alpha = 5.0$, $z_g = 1500$, and $G = 0.8$
Substituting into the first equation in Note 2 yields

$$K_z = 2.10\,(Z/Z_g)^{2/\alpha} = 1.39$$

Substituting into Equation (65) yields

$$Q_z = 0.00256 \times 1.39 \times (120)^2 \times 1.15 = 51.2 \text{ lb/ft}^2$$

Substituting into Equation (64) yields the design force wind as

$$F_W = 51.2 \times 0.8 \times 1.0 \times 224 = 9175 \text{ lb}$$

Example 9. Church in Key West, Florida. The top of the tower is 50 ft above ground level.

Solution:

From Figure 11, the design speed is found to be 150 mph

Table 20 Classification of Buildings and Other Structures for Wind Loads

Nature of Occupancy	Category
Buildings and other structures that represent a low hazard to human life in event of failure, including, but not limited to, agricultural facilities, certain temporary facilities, and minor storage facilities	I
All buildings and other structures except those listed in Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in event of failure, including, but not limited to, - Buildings and other structures where more than 300 people congregate in one area. - Buildings and other structures with elementary and secondary schools, day care facilities with capacity greater than 250 - Buildings and other structures with capacity greater than 500 for colleges or adult education facilities - Health care facilities with capacity of 50 or more resident patients, but not having surgery or emergency treatment facilities - Jails and detention centers - Power generating stations and other public utility facilities not included in Category IV - Buildings and others structures containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released	III
Buildings and other structures designated as essential facilities including, but not limited to, - Hospitals and other health care facilities with surgery and emergency treatment facilities - Fire, rescue, and police stations and emergency vehicle garages - Designated earthquake, hurricane, or other emergency shelters - Communication center and other facilities required for emergency response - Power generating stations and other public utility facilities required in an emergency - Buildings and other structures with critical national defense functions	IV

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From Table 16, use Category III

From Table 17, use Exposure D

From Table 18, $I = 1.15$

From Table 20, G = 0.85

From Table 21, $K_z = 1.27$

From Equation (65):

$$Q_z = 0.00256 \times 1.27 \times 1.15 \times (150)^2 = 84.1 \text{ lb/ft}^2$$

Substituting into Equation (64) gives the design wind force as

$$F_w = 84.1 \times 0.85 \times 1.15 \times 224 = 18{,}415 \text{ lb}$$

Table 21 Velocity Pressure Exposure Coefficient K_z

Height above ground level z, ft	Exposure A	Exposure B	Exposure C	Exposure D
0 to 15	0.32	0.57	0.85	1.03
20	0.36	0.62	0.90	1.08
25	0.39	0.66	0.94	1.12
30	0.42	0.70	0.98	1.16
40	0.47	0.76	1.04	1.22
50	0.52	0.81	1.09	1.27
60	0.55	0.85	1.13	1.31
70	0.59	0.89	1.17	1.34
80	0.62	0.93	1.21	1.38
90	0.68	0.96	1.24	1.40
100	0.68	0.99	1.26	1.43
120	0.73	1.04	1.31	1.48
140	0.78	1.09	1.36	1.52
160	0.82	1.13	1.39	1.55
180	0.86	1.17	1.43	1.58
200	0.90	1.20	1.46	1.61
250	0.98	1.28	1.53	1.68
300	1.05	1.35	1.59	1.73
350	1.12	1.41	1.64	1.78
400	1.18	1.47	1.69	1.82
450	1.24	1.52	1.73	1.86
500	1.29	1.56	1.77	1.89

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Notes:

1. Linear interpolation for intermediate values of height z is acceptable.
2. For values of height z greater than 500 ft, K_Z must be calculated using the following equations:

$$K_Z = 2.01(z/z_g)^{2/\alpha} \quad \text{For } 15 \text{ ft} \le z \le z_g$$

or

$$K_Z = 2.01(15/z_g)^{2/\alpha} \quad \text{For } z < 15 \text{ ft}$$

3. Exposure categories are defined in Table 14.
4. Values for alpha (α) and z_g are found in Table 17.

REFERENCES

AISC. 1989. *Manual of steel construction—Allowable stress design*, 9th ed. American Institute of Steel Construction, Chicago.

ASCE. 1995. Minimum design loads for buildings and other structures. *Standard* ASCE 7-95. American Society of Civil Engineers, Reston, VA.

ASHRAE. 2000. A practical guide to seismic restraint. Research Project RP-812, *Final Report*.

ASME. 2003. Nuclear air and gas treatment. Code AG-1-2003. American Society of Mechanical Engineers, New York.

ASTM. 1996. Test methods for strength of anchors in concrete and masonry elements. *Standard* E488-96 (R2003). American Society for Testing and Materials, West Conshohocken, PA.

ATC. *Proceedings of seminar on seismic design, performance, and retrofit of nonstructural components on critical facilities*. ATC 29-2. Applied Technology Council, Washington, D.C.

AWC. 1997. *National design specification (NDS®) for wood construction*. American Wood Council, Washington, D.C.

BOCA. 1996. *The BOCA National building code*, 13th ed. Building Officials & Code Administrators International, Inc., Country Club Hills, IL.

Cover, L.E., et al. 1985. Handbook of nuclear power plant seismic fragilities. *Report* NUREG/CR-3558. Lawrence Livermore National Laboratory and U.S. Nuclear Regulatory Commission, Washington, D.C.

DOD. 1990. Structures to resist the effects of accidental explosions. *Technical Manual* TM 5-1300. U.S. Department of Defense, Washington, D.C.

DOD. 2002. Design and analysis of hardened structures to conventional weapons effects. *Technical Manual* TM 5-855-1. U.S. Department of Defense, Washington, D.C.

ICC. 2000. *International building code*. International Code Council, Falls Church, VA.

ICBO. 1997. *Uniform building code*. International Conference of Building Officials, Whittier, CA. (Now part of ICC.)

SBCCI. 1994. *Standard building code* 1996. Southern Building Code Congress International, Inc., Birmingham, AL.

SMACNA. 1998. *Seismic restraint manual: Guidelines for mechanical systems*, 2nd ed. Sheet Metal and Air Conditioning Contractors' National Association, Chantilly, VA.

U.S. Army, Navy, and Air Force. 1992. *Seismic design for buildings*. TM 5-809-10, NAVFAC P-355, AFN 88-3, Chapter 13.

BIBLIOGRAPHY

ACI. 1995. Building code requirements for structural concrete. *Standard* 318-95 and *commentary* 318R-95. American Concrete Institute, Farmington Hills, MI.

AISC. 1995. *Manual of steel construction—Load and resistance factor design*, 2nd ed. American Institute of Steel Construction, Chicago.

Associate Committee on the National Building Code. 1985. *National building code of Canada* 1985, 9th ed. National Research Council of Canada, Ottawa.

Associate Committee on the National Building Code. 1986. *Supplement to the National Building Code of Canada* 1985, 2nd ed. National Research Council of Canada, Ottawa. First errata, January.

ATC. *Proceedings of seminar and workshop on seismic design and performance of equipment and nonstructural elements in buildings and industrial structures*. ATC 29, NCEER (New York) & NSF (Washington D.C.).

ATC. *Seminar on seismic design, retrofit, and performance of nonstructural components*. ATC 29-1, NCEER (New York) & NSF (Washington D.C.).

AWS. 2000. *Structural welding code*. AWS D1.1-2000. Steel American Welding Society, Miami.

Ayres, J.M. and R.J. Phillips. 1998. Water damage in hospitals resulting from the Northridge earthquake. *ASHRAE Transactions* 104(1B):1286-1296.

Batts, M.E., M.R. Cordes, L.R Russell, J.R. Shaver, and E. Simiu. 1980. *Hurricane wind speeds in the United States.* NBS BSS 124. National Institute of Standards and Technology, Gaithersburg, MD.

Bolt, B.A. 1988. *Earthquakes*. W.H. Freeman, New York. DOE. 1989. General design criteria. DOE Order 6430.1A. U.S. Department of Energy, Washington, D.C.

FEMA 368 & 369. *NEHRP recommended provisions for seismic regulations for new buildings and other structures. Part 1, Provisions; Part 2, Commentary.* Building Seismic Safety Council, Washington, D.C.

ICC-ES. 2007. *Acceptance criteria for seismic qualification by shake-table testing of nonstructural components and systems.* AC156. ICC Evaluation Service, Inc., Whittier, CA.

Jones, R.S. 1984. Noise and vibration control in buildings. McGraw-Hill, New York.

Kennedy, R.P., S.A. Short, J.R. McDonald, M.W. McCann, and R.C. Murray. 1989. *Design and evaluation guidelines for the Department of Energy facilities subjected to natural phenomena hazards*.

Lama, P.J. 1998. Seismic codes, HVAC pipe systems and practical solutions. *ASHRAE Transactions* 104(1B):1297-1304.

Maley, R., A. Acosta, F. Ellis, E. Etheredge, L. Foote, D. Johnson, R. Porcella, M. Salsman, and J. Switzer. 1989. Department of the Interior, U.S. geological survey. U.S. geological survey strong-motion records from the Northern California (Loma Prieta) earthquake of October 17, 1989. Open-file *Report* 89-568.

Meisel, P.W. 2001. Static modeling of equipment acted on by seismic forces. *ASHRAE Transactions* 107(1):775-786.

Naeim, F. 1989. *The seismic design handbook*. Van Nostrand Reinhold International Company Ltd., London, England.

Naeim, F. 2001. *The seismic design handbook,* 2nd ed. Kluwer Academic, Boston.

NFPA. 2002. *Installation of sprinkler systems*. National Fire Protection Association, Quincy, MA.

Peterka, J.A., and J.E. Cermak. 1974. Wind pressures on buildings—Probability densities. *J. Structural Div.*, ASCE 101(6):1255-1267.

Simiu, E., M.J. Changery, and J.J. Filliben. 1979. *Extreme wind speeds at 129 stations in the contiguous United States*. U.S. NBS BSS 118. National Institute of Standards and Technology, Gaithersburg, MD.

SMACNA. 1995. *HVAC duct construction standard—metal and flexible*, 2nd ed. Sheet Metal and Air Conditioning Contractors' National Association, Chantilly, VA.

Wasilewski, R.J. 1998. Seismic restraints for piping systems. *ASHRAE Transactions* 104(1B):1273-1295.

Weigels, R.L. 1970. *Earthquake engineering,* 10th ed. Prentice-Hall, Englewood Cliffs, NJ.