

SEISMIC AND WIND RESTRAINT DESIGN

<i>SEISMIC RESTRAINT DESIGN</i>	54.1	<i>Weld Capacities</i>	54.12
<i>Terminology</i>	54.2	<i>Seismic Snubbers</i>	54.12
<i>Calculations</i>	54.2	<i>Examples</i>	54.13
<i>Applying Static Analysis</i>	54.5	<i>Installation Problems</i>	54.19
<i>Computation of Loads</i>	54.5	<i>WIND RESTRAINT DESIGN</i>	54.19
<i>Lag Screws into Timber</i>	54.11	<i>Terminology</i>	54.20
<i>Anchor Bolts</i>	54.12	<i>Calculations</i>	54.21

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 the movement of equipment and to keep the equipment captive during an earthquake or during 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.

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) has developed standards on restraint design for fire protection systems.
- 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 812), published by ASHRAE (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

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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 is generally on the order of four times the acceleration of gravity (4g) 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 zones. 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. Such 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

A dynamic analysis may be based on site-specific ground motions developed by a geotechnical or soils engineer. A common approach assumes an elastic response spectrum. The results of the dynamic analysis are then scaled up or down as a percentage of the total lateral force obtained from the static analysis performed on the building.

The scaling coefficient is established by the applicable code or by the governing building official. The scaled acceleration calculated by the structural engineer at any level in the structure can be determined and compared to the force calculated in Equation (1). The greater of the two should be used in the anchor design. (*Note:* The IBC does not have provisions for scaling the results.)

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 will increase. Computer programs have been developed to evaluate the system and provide output results in the form of applied loads to supports, total deformation of the system, and internal system stresses. This analysis typically exceeds the required static analysis allowed by the building codes.

Establishing Design Static Force

This chapter details code requirements for the 2000 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 new 2000 IBC design equations, the design force should be determined based on criteria in the 1997 UBC.

A section is also included for calculation using the 1994 UBC, which is the basis of the design requirements for OSHPD applications in California.

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 IBC specifies a design lateral force F_p for nonstructural components as

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

but F_p need not be greater than

$$F_p = 1.6 S_{DS} I_p W_p \quad (2)$$

nor less than

$$F_p = 0.3 S_{DS} I_p W_p \quad (3)$$

$$S_{DS} = 2F_a S_S / 3 \quad (4)$$

where

a_p = component amplification factor in accordance with [Table 1](#).
 S_{DS} = design spectral response acceleration at short periods. S_S is the mapped spectral acceleration from [Figure 1](#) and $0.8 \leq F_a \leq 2.5$. (*Note:* More detailed maps for the United States are available at the U.S. Geological Survey website: 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 2](#). (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 1](#). (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 (see [Table 3](#)).

$1 + 2z/h$ = 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 $z \geq 0$ and z/h need not exceed 1.

Table 1 Coefficients for Mechanical Components (2000 IBC)

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

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.

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

Table 2 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					

^aUse straight-line interpolation for intermediate values of mapped spectral acceleration at short period S_s .

^bSite-specific geotechnical investigation and dynamic site response analyses must be performed to determine appropriate values.

^cD is the default Site Class unless otherwise stated in the approved geotechnical report.

Table 3 Seismic Groups and Building Importance Factors (2000 IBC)

Building Seismic Importance Group	Building Factor	Nature of Occupancy
I	1.0	Buildings and other structures except those listed in Groups II, III, IV
II	1.25	<ol style="list-style-type: none"> Structures where more than 300 people congregate in one area Structures with elementary, secondary school or day care facilities with a capacity of over 250 Structures with a capacity greater than 500 for colleges or adult education facilities Health care facilities with a capacity of 50 or more resident patients but no surgery or emergency treatment facilities Jails and detention facilities Any other occupancy with an occupant load greater than 5000 Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Group III 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	<ol style="list-style-type: none"> Hospitals and other health care facilities with surgery or emergency treatment facilities Fire, rescue, and police stations and emergency vehicle garages Designated earthquake, hurricane, or other emergency shelters Designated emergency preparedness, communication and operation centers and other facilities required for emergency response Power-generating stations and other public utility facilities required as emergency backup facilities for Group III structures Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the exempt amounts of Table 307.7(2) Aviation control towers, air traffic control centers and emergency aircraft hangers Structures with critical national defense functions 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

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

but F_p need not be greater than

$$F_p = 4C_a I_p W_p \quad (6)$$

nor less than

$$F_p = 0.7C_a I_p W_p \quad (7)$$

where

a_p = component amplification factor in accordance with Table 4

C_a = seismic coefficient (determined from Figure 2 and Tables 5, 6, 7, and 10 as appropriate)

R_p = component response modification factor in accordance with Table 4

I_p = component importance factor in accordance with Table 9

$1 + \frac{3h_x}{h_r}$ = height amplification factor where h_x is the height of attachment in the structure and h_r is the average height of the 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 the 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.)

Table 4 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

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Table 5 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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The total design lateral seismic force is given as

$$F_p = Z I_p C_p W_p \quad (8)$$

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 5. *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.

Table 6 Seismic Coefficient C_a (1997 UBC)

Soil Profile Type	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
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				

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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 7 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

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Notes: 1. Interpolation between tabulated values is permitted.

2. Source type must be as determined by approved geological data.

Table 8 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.

Table 9 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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The horizontal force factor C_p is determined from Table 8 based on the type of equipment, the tie-down configuration, and the type of base.

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

APPLYING STATIC ANALYSIS

The forces acting on a piece of equipment are the vertical and lateral forces resulting from the earthquake, the force of gravity, and the 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.

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. Although there are many different valid methods that can be used to

determine these loads, it is the intent of this section to suggest 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 the equipment has been shake 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

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

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

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

when using allowable strength based design factors.

where

$$D = \text{dead load affect}$$

$$S_{DS} = \text{short period spectral response (0.2 sec)}$$

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.

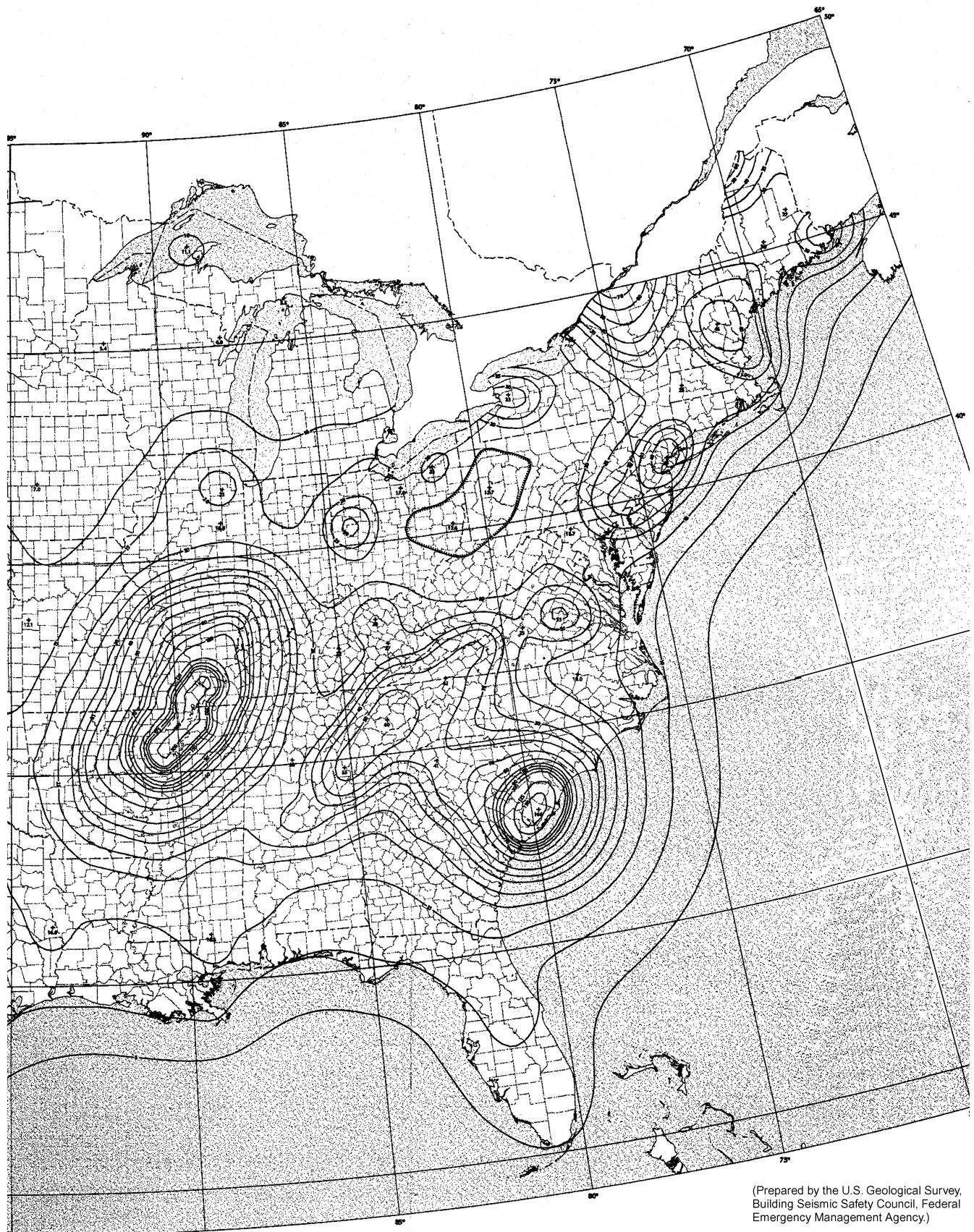


Fig. 1B Maximum Considered Earthquake Ground Motion for the United States (Concluded)

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>)

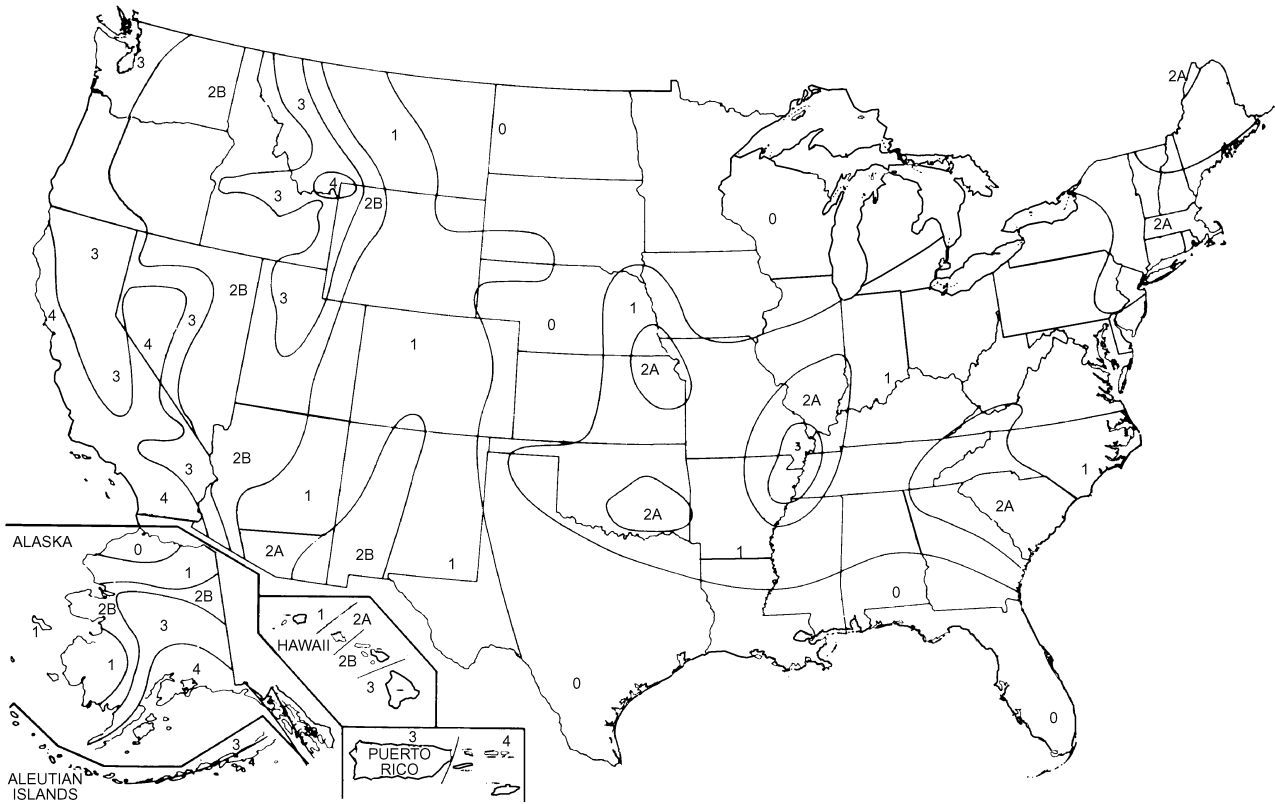


Fig. 2 Seismic Zone Map of the United States (1994 and 1997 UBC)

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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} = 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:

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

Thus,

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

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

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

The 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.

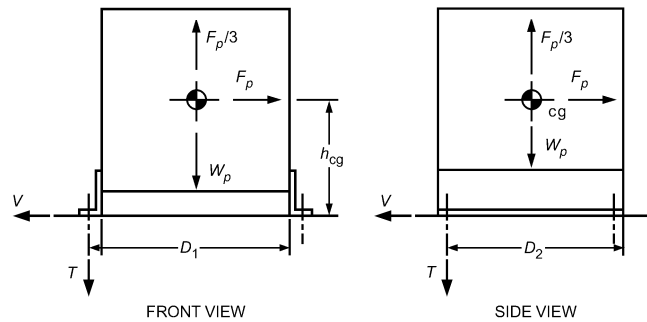


Fig. 3 Equipment with Rigidly Mounted Structural Bases

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

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

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

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

The allowable forces T_{allow} and V_{allow} are the generic allowable capacities given in Table 11 for wedge anchor bolts.

Table 10 International Seismic Zones (1994 and 1997 UBC)

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

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Table 11 Typical Allowable Loads for Wedge Anchors

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 ICBO reports.

General Case

The classic method used to distribute seismic loads equally distributes the 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 the 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 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 later in this chapter.)

Determination of Vertical Seismic Forces at Restraints. The 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 will be addressed here as a separate entity.

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

$$T_n = \pm W_n F_{pv}/W_p \quad (18)$$

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

$$\theta = \arctan (B/A) \quad (19)$$

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

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

and along the short axis is

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

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

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

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

$$T_{eff} = -W_n + T_n + T_m \quad (23)$$

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

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

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.

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

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

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

$$T_m = T_{mx} + T_{my} \quad (27)$$

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

$$T_n = \pm W_n F_{pv}/W_p \quad (28)$$

and the total vertical force component is

$$T_{eff} = -W_n + T_m + T_n \quad (29)$$

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 to be 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 impact that a reduction in weight 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 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

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

(where S_v is the shear stress in the bolt in psi). T_{allow} , the maximum allowable tensile stress, is not to 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 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 4](#). 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

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

where

$$\begin{aligned} T_{eff} &= \text{tensile load at anchor} \\ T_{allow} &= \text{maximum allowable anchor tensile load} \\ V_{eff} &= \text{shear load at anchor} \\ V_{allow} &= \text{maximum allowable anchor shear load} \end{aligned}$$

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 pre-loaded 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 $0.707 \times 1/16 = 0.0442$ 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:

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

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 if necessary.
- 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 (see Figure 4). Many devices are presently approved with Office of Statewide Health Planning Development (OSHPD) ratings.

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 the 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 the 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.

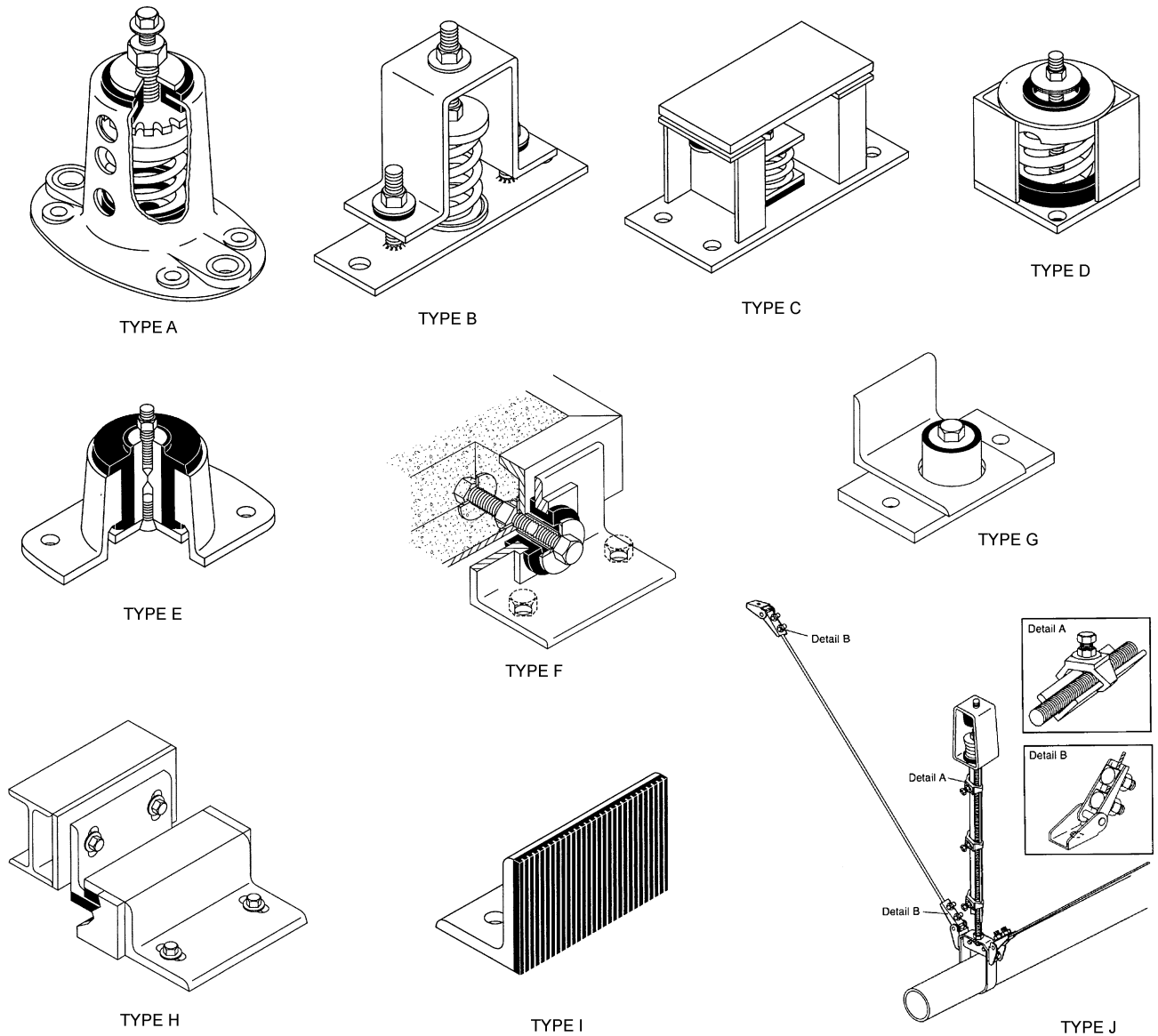


Fig. 4 Seismic Snubbers

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 will be initially determined using the 2.5 factor. For attach-

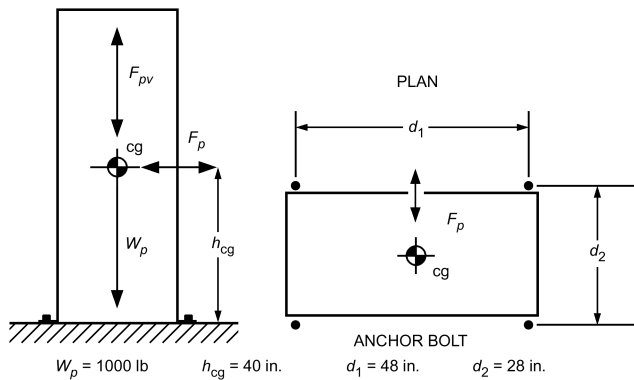


Fig. 5 Equipment Rigidly Mounted to Structure (Example 1)

ment to concrete with post-installed anchors, the 1.5 factor will be used and an embedment depth of over 8 bolt diameters is assumed. For rigidly mounted mechanical equipment, a_p from Table 1 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 2, site class C):

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

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

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

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

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.

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

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

Calculate the overturning moment (OTM):

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

Calculate the resisting moment (RM):

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

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

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

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

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.)

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 $\alpha = \arcsin (T_{eff} / Z'_\alpha) = 42.5^\circ$, where Z'_α 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.

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.

$$\text{From Table 11, } T_{allow} = 600 \text{ lb and } V_{allow} = 1200 \text{ lb}$$

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.

Case 3. Equipment attached to steel

For equipment attached directly to a steel member, the analysis is the same as that shown in Case 1 above. The allowable values for the attaching bolts are given in the *Manual of Steel Construction* (AISC

Table 12 Allowable Loads for A307 Bolts (per ASD)

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

1989). Values for A307 bolts are given in Table 12. 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 given 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 $A_b = 0.196$.

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

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 (see 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 1 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

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

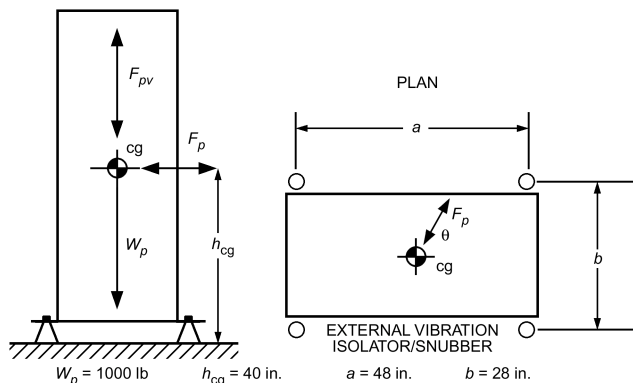


Fig. 6 Equipment Supported by External Spring Mounts

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 $1869 \times 2 = 3738 \text{ lb}$ and F_{pv} is $125 \times 2 = 250 \text{ lb}$.

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 $dT/d\theta = 0$:

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

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

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

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

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

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

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 = distance from edge of isolator base plate to center of bolt hole.

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

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

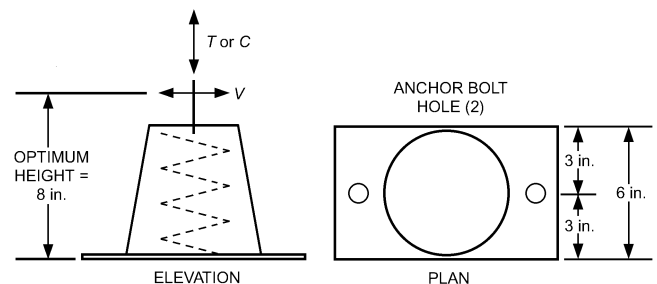


Fig. 7 Spring Mount Detail (Example 2)

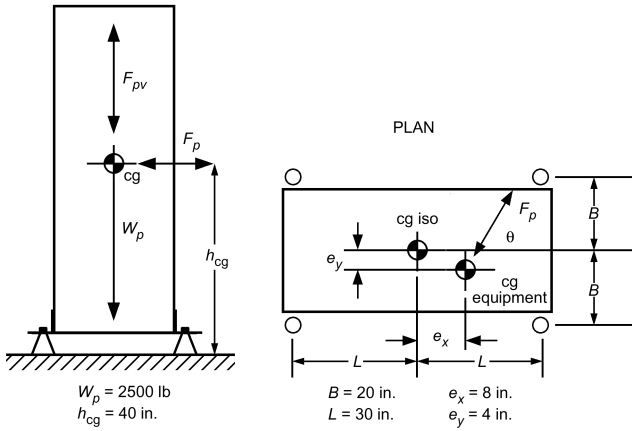


Fig. 8 Equipment with Center of Gravity Different from Isolator Group (in Plan View)

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

Determine whether 5/8 in. post-installed anchors *with special inspection* will handle this load. From Equation (15) and Table 7,

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

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 (see Figure 8).

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

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

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

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

$$\phi = \tan^{-1}(L I_x / B I_y)$$
 (43)

Vertical reactions

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

Vertical reaction due to overturning moment

$$T_m = \pm F_p \left(\frac{B h_{cg}}{I_x} \cos \phi + \frac{L h_{cg}}{I_y} \sin \phi \right)$$
 (45)

Vertical reaction due to eccentricity

$$(T_e)_{max/min} = (W_n)_{max/min} \left(\frac{B e_y}{I_x} + \frac{L e_x}{I_y} \right)$$
 (46)

Vertical reaction due to W_p

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

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

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

Horizontal reactions

Horizontal reaction due to rotation

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

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

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

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

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),

$$\theta = 33.69^\circ \quad \alpha = 63.43^\circ$$

$$\beta = 150.26^\circ \quad \phi = 33.69^\circ$$

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),

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

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

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

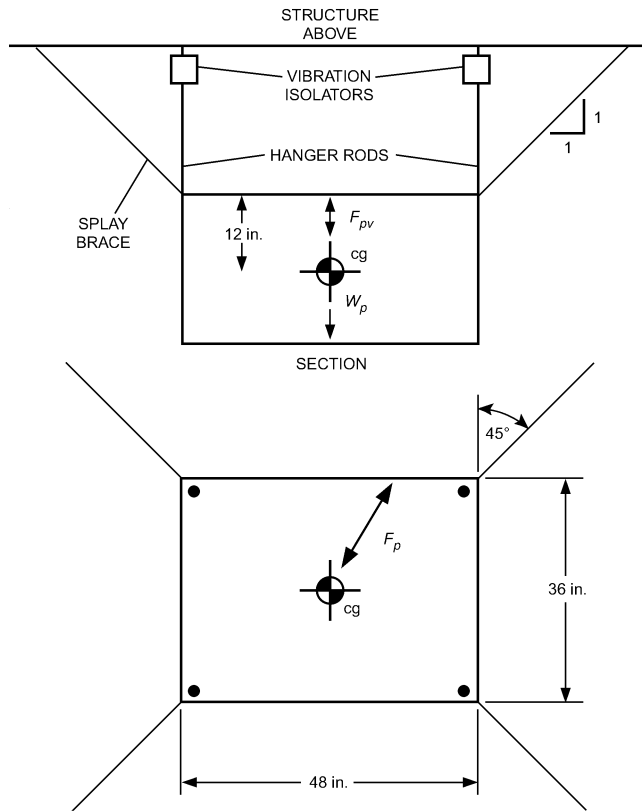
The values of T_{min} and V_{max} are used to design the anchorage of the isolators and/or snubbers, and T_{max} is used to verify the adequacy of the structure to resist the vertical loads.

Example 4. Anchorage design for equipment with supports and bracing for suspended equipment (see Figure 9). Equipment weight $W_p = 500 \text{ lb}$.

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

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



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

Fig. 9 Supports and Bracing for Suspended Equipment

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

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

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

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

Angle $\phi = \tan^{-1}(L_x / B I_y) = 36.86^\circ$

From Equation (44),

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

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:

$$\text{Maximum tensile} = 389 \text{ lb}$$

$$\text{Maximum compression} = 139 \text{ lb}$$

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

Due to 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.

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 the 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

$$\text{Force in the slack cable} = 1587 \text{ lb}$$

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

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

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

From Table 11,

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

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.

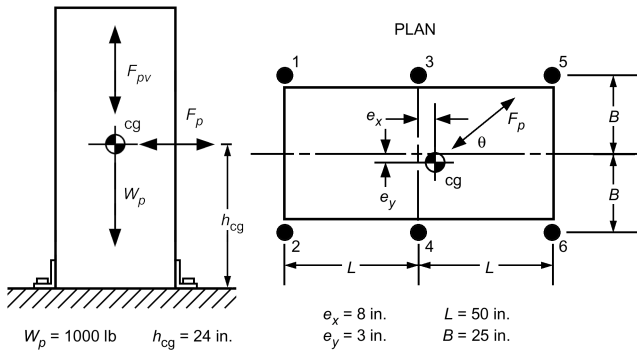


Fig. 10 Floor-Mounted Equipment with Solid or Isolated Connection to Structure (Example 5)

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 (see 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

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

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

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

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

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

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

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

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

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

The equation for calculating the maximum loads at any point in this system due to 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

$$f_b = \frac{(M_x I_y - M_y I_{xy})L}{I_x I_y - I_{xy}^2} + \frac{(M_x I_x - M_y I_{xy})B}{I_x I_y - I_{xy}^2}$$

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

From Equation (43),

$$\phi = \tan^{-1}(L I_x / B I_y) \text{ and therefore } \theta = \tan^{-1}(L I_x / B I_y)$$

$$\phi = \theta = 36.87^\circ$$

From Equation (44),

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

From Equation (45) and setting angle $\theta =$ 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,

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

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 g s 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 $\phi = 30^\circ$

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

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

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

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

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 impact 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.

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

and

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

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

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

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

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

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

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

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 manufacturer's 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 upon 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.

ASCE *Standard 7-95, Minimum Design Loads for Buildings and Other Structures*, 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 (ASCE 7-98) includes more comprehensive and rigorous procedures for evaluating wind forces and wind restraint.

TERMINOLOGY

Classification. Buildings and other structures are classified for wind load design according to [Table 13](#).

Table 13 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 the 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 the event of failure including, but not limited to, <ul style="list-style-type: none"> - 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 a capacity greater than 500 for colleges or adult education facilities - Health care facilities with a 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, <ul style="list-style-type: none"> - 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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Basic wind speed. The fastest mile-per-hour wind speed at 33 ft above the ground of Terrain Exposure C (see [Table 14](#)) 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 have not been 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 (see [Table 15](#)). 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.

Table 14 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 in the upwind direction for a distance of 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 due to the building or structure being located 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 having the size of single-family dwellings or larger. Use of this exposure category shall be limited to those areas for which terrain representative of Exposure B prevails in the upwind direction for a distance of 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 shall apply only to those buildings and other structures exposed to the wind coming from over the water. Exposure D extends inland from the shoreline a distance of 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 a mean roof height of 60 ft or less are designed on the basis of Exposure C.
2. HVAC components on buildings with a mean roof height greater than 60 ft and other structures are designed on the basis of the exposure categories defined in this table except assume Exposure B for buildings and other structures sited in terrain representative of Exposure A.

Table 15 Wind Importance Factor

Importance Factor, I (Wind Loads)	
Category	I
I	0.87
II	1
III	1.15
IV	1.15

Minimum design wind load. The wind load may not be less than 10 lb/ft² 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 analysis method for standard component shapes. The second method, the **wind-tunnel procedure**, is incorporated in the analysis of complex and unusually shaped components or equipment that are located on sites that produce wind channeling or buffeting due to 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/ft²

- G = gust response factor for HVAC components evaluated at height z above ground level
- C_f = force coefficient (Table 16)
- A_f = area of HVAC component projected on a plane normal to wind direction, ft²

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 17
- V = velocity from Figure 11, mph
- I = importance factor from Table 15

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

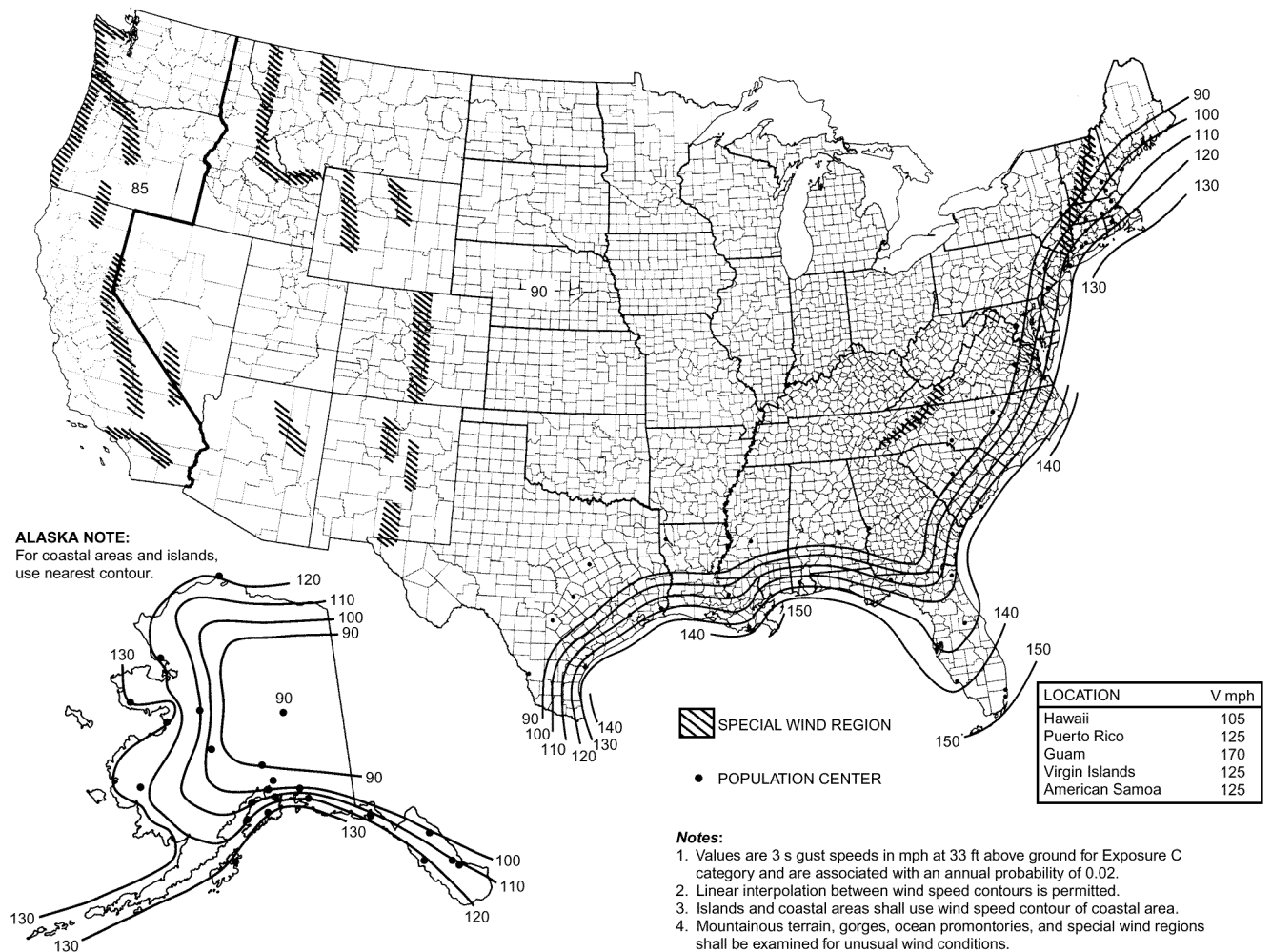


Fig. 11 Wind Speed Data
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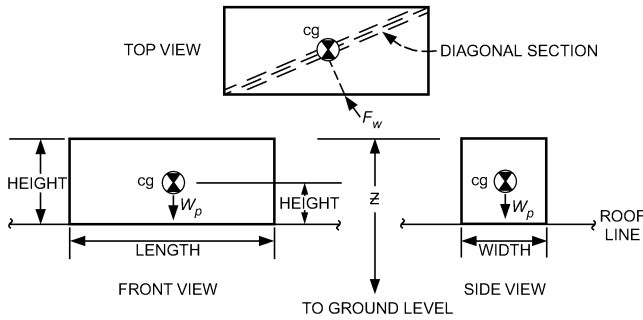


Fig. 12 Equipment Dimensions and Force Locations for Wind Examples 7, 8, and 9

Table 16 Force Coefficients for HVAC Components, Tanks, and Similar Structures

Shape	Type of Surface	C_f for h/D Values of		
		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 ($D\sqrt{Q_z} \leq 2.5$)	All	0.7	0.8	1.2

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Notes:

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

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 = $A_f = 10 \times 22.4 = 224$ ft²
 From Table 16; $C_f = 1.0$ for wind acting along diagonal with $h/D = 10/10 = 1$.

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 13, use Category IV
 From Table 14, use Exposure A
 From Table 15, $I = 1.15$
 From Table 18, $K_z = 0.68$
 From Table 17, $G = 0.8$

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

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

Table 17 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 14 for definitions of Exposure Categories.

Table 18 Velocity Pressure Exposure Coefficient K_z

Height above ground level, z , ft	Exposure			
	A	B	C	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:

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

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

or

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

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

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 13, use Category II.
 From Table 14, use Exposure A
 From Table 15, $I = 1.0$
 Because $z > 500$ ft, K_z must be determined from Note 2 of Table 18.
 From Table 17, $\alpha = 5.0$, $z_g = 1500$, 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

From [Table 13](#), use Category III

From [Table 14](#), use Exposure D

From [Table 15](#), $I = 1.15$

From [Table 17](#), $G = 0.85$

From [Table 18](#), $K_z = 1.27$

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

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

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