Related Commercial Resources

CHAPTER 55

SEISMIC- AND WIND-RESISTANT DESIGN

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

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

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

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

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

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

The *International Building Code*® (IBC) (ICC 2009) provides a prescriptive approach for applying an equivalent static force representing the dynamic forces transmitted to the equipment by seismic or high-wind events. For mechanical systems, analysis of seismic and wind loading conditions can use static analysis from the prescriptive approach. 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, lag bolts, welds, or concrete anchors. The substructure must be capable of surviving the imposed forces.

1. SEISMIC-RESISTANT DESIGN

Most seismic requirements adopted by local jurisdictions in North America are based on model codes developed by the International Code Council (ICC), such as the *International Building Code* (IBC). The *National Building Code of Canada* (NRC-IRC 2010) is Canada's equivalent version of the 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* (SMACNA 2008), includes seismic restraint information for mechanical equipment subjected to seismic forces of up to 1.0g.
- The most current National Fire Protection Association (NFPA) standards on restraint design are compliant with the IBC.
- U.S. Department of Energy DOE 430.1A and ASME AG-1 cover restraint design for nuclear facilities.
- DOD (2007) provides guidance for seismic design for U.S. Department of Defense (DoD) and Department of State (DoS) facilities, and DOD (2005) provides the seismic and wind design constants.
- *A Practical Guide to Seismic Restraint* (ASHRAE 2000) covers a broad range of seismic restraint design issues.
- *Federal Emergency Management Agency (FEMA)* installation manuals FEMA-412, FEMA-413, and FEMA-414 are available at the FEMA Web site. They provide a step-by-step process with details for installing seismic restraint devices.

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 applicable to the project), selects the required vibration isolation devices. Seismic restraint calculations are performed for nonstructural components, and designs for piping, ductwork, and conduits are designed and detailed. For design-build projects, the design is reviewed by the registered design professional. Nonstructural restraint components are designed and constructed to resist the aftereffects of earthquake motions as required by the applicable code

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and in accordance with local building officials. Reviewed designs are submitted for approval by the authority having jurisdiction.

To ensure proper design factors are used, a designer should obtain information on the seismic design conditions (site class and occupancy category). The importance of the equipment and systems affected should be understood for code applications to include those items that must be functional after seismic events.

The owner or building officials maintain the code-required quality control over the design by requiring construction documents, special inspection requirements, and certification requirements prepared by the registered design professional and approved by the authority having jurisdiction. Upon completion of installation, the supplier of the seismic restraints, or a qualified representative, should inspect the installation and verify that all restraints and force-resisting systems are installed properly and comply with specifications.

1.1 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.

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. Horizontal force generated at the plane of the seismic restraints, acting to cut the restraint at the base.

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

Seismic force levels. The geographic location of a facility determines its seismic spectral response acceleration levels, as given in the *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.

1.2 CALCULATIONS

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

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

Dynamic Analysis

Dynamic analysis of the isolation and snubber systems may be based on ground-level response spectra given in the IBC and reference standard ASCE 7 (ASCE 2005), which can be used as input for a dynamic analysis.

Response spectra applied to nonstructural components can be developed from ICC-ES acceptance criteria AC 156 (ICC-ES 2007). Site-specific ground response spectra developed by a geotechnical or soils engineer may be used, as well. The computer analysis used must be capable of analyzing nonlinear supports and site-specific ground motions. This dynamic analysis provides the maximum seismic input accelerations to the equipment components, allowing comparison to three-dimensional shock (drop) or shaker test fragility levels to determine equipment survivability. Actual drop or shaker test data for all HVAC equipment may not be available for the next several years.

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

Under some conditions, Chapter 17 of IBC requires certificates of compliance for components and their attachments for a component importance factor I_p of 1.0 or 1.5. This is a life-safety issue as well as an essential equipment issue. Essential equipment with an $I_p = 1.5$ must have a certificate of compliance. Issuance of a certificate of compliance to the engineer of record and building official can be based on dynamic analysis. Most building officials require a stamp by a registered professional to be part of the calculations and certificate of compliance. Table 1 provides guidance on type of analysis (static or dynamic) and certificate of compliance documentation is required. Sample dynamic analysis is beyond the scope of this chapter and should be provided by experienced registered professionals. A common approach assumes an elastic response spectrum. The results of the dynamic analysis can then be scaled up or down as a percentage of the total lateral force obtained from the static analysis performed on the building.

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

Static Analysis as Defined in the *International Building Code*

The IBC specifies a design lateral force F_p for nonstructural components as

$$F_p = (0.4 a_P S_{DS} W_P)\frac{I_p}{R_p}\left(1 \quad 2\frac{Z}{h}\right) \tag{1}$$

but F_p need not be greater than

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

nor less than

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

where S_{DS} is determined by

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

where

a_p = component amplification factor in accordance with Table 2.
S_{DS} = design spectral response acceleration at short periods. S_S is the mapped spectral acceleration from Tables 4 and 5. (*Note*: More

Table 1 IBC Seismic Analysis Requirements

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

*If in question, reference structural documents.

Table 2 Coefficients for Mechanical Components

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: IBC (2006).

detailed maps for the United States are available at the U.S. Geological Survey Web site: www.usgs.gov.)

F_a = function of site soil characteristics and must be determined in consultation with either project geotechnical (soils) or structural engineer. Values for F_a for different soil types are given in Table 3. (*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 IBC.

I_p = component importance factor (see the IBC for explanation and determination of I_P).

$1 + 2z/h$ = 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.

$W_p(D)$ = mass of equipment, which includes all items attached or contained in the equipment.

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; these act on the center of gravity. 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 (for the IBC) to be

$$F_{pv} = 0.2 S_{DS} D \tag{5}$$

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 7.46 kW are undercut anchors.

Tables 4 and 5 contain brief listings of S_s factors that can be used to calculate the magnitude of the horizontal static seismic force acting at the equipment center of gravity. Values for IBC 2006 are available on the USGS web site for U.S. locations or in Tables F-2 and G-2 of DOD (2005) for worldwide locations.

Table 3 Values of Site Coefficient F_a as Function of Site Class and Spectral Response Acceleration at Short 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 IBC for more information					

[a] Use straight-line interpolation for intermediate values of mapped spectral acceleration at short period S_s.

[b] Site-specific geotechnical investigation and dynamic site response analyses must be performed to determine appropriate values.

[c] D is the default Site Class unless otherwise stated in the approved geotechnical report.

1.3 APPLYING STATIC ANALYSIS

The prescriptive method in the IBC allows that an equivalent static force can be calculated that represents the dynamic motions of an earthquake. The static forces acting on a piece of equipment are vertical and lateral forces resulting from the earthquake, the force of gravity, and forces at the restraints that hold the equipment in place. The analysis assumes that the equipment does not move during the earthquake and that the relative accelerations between its center of gravity and the ground generate forces that must be balanced by reactions at the restraints. Guidance from the code bodies indicates that equipment can be analyzed as though it were a rigid component; however, a factor a_p is applied in 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

Table 4 S_s Numbers* for Selected U.S. Locations (U.S. COE 1998)

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

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

forces at all restraint points for any possible direction that the lateral wave front can follow to ensure that the attachment is adequate.

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

Under some instances (particularly those relating to life-support issues in hospital settings), newer code requirements indicate that critical equipment must be seismically qualified to ensure its

Table 5 S_s Numbers for Selected International Locations (U.S. COE 1998)

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

Table 6 Load Combinations
(Equation Numbers as Referenced in IBC)

ASD	LRFD
5. $(1.0 + 0.14S_{DS})D + H + F + 0.7\rho Q_E$	5. $(1.2 + 0.2S_{DS})D + \rho Q_E + L + 0.2S$
8. $(0.6 + 0.14S_{DS})D + 0.7\rho Q_E + H$	7. $(0.9 - 0.2S_{DS})D + \rho Q_E + 1.6H$

continued operation during and after a seismic event. Special care must be taken in these situations to ensure that equipment has been shaker tested or otherwise certified to meet the maximum anticipated seismic load. Table 6 illustrates some load combination calculations.

1.4 COMPUTATION OF LOADS AT BUILDING CONNECTION

ASCE *Standard* 7 is based on **load- and resistance-factor design (LRFD)**. In the past, building codes have been based on **allowable stress design (ASD)**. Both are allowed for seismic restraint design. Load factors and load combinations that must be considered in design are defined in Chapter 2 of ASCE *Standard* 7. If a component is anchored with post-installed anchors, the design is usually accomplished using provisions of LRFD.

The Load Combinations of Section 2.4.1 of ASCE *Standard* 7 must be considered in the design. Generally, for rigidly mounted components, Combination 8 is the critical combination to be considered.

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 1. All bolts or as few as a single bolt may be affected.

Simple Case

Figure 1 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

$$0 = F_p - V \qquad (6)$$

The equipment shown in Figure 1 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 results in an overturning moment (OTM) and resisting moment (RM).

For Figure 1, two bolts are in tension. See Example 1 for applications of the OTM and RM. See ASCE *Standard* 7 for load combinations that adjust the *D* (dead load) and *E* (earthquake load). Shear and tension forces *V* and *T* should be calculated independently for both axes, as shown in the front and side views. See the examples for complete analysis.

General Case

The classic method used to distribute seismic loads equally distributes lateral loads among the restraints and then modifies these loads as a function of the mass eccentricity. Worst-case mass, vertical seismic load, and overturning components are combined to determine a maximum vertical load component. This **polar method** is in common use and works well for most applications.

A second, **lump mass method**, proportions the restraint loads based on the equipment mass 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. 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. Eccentric loads can be addressed with either the polar method or the lump mass method.

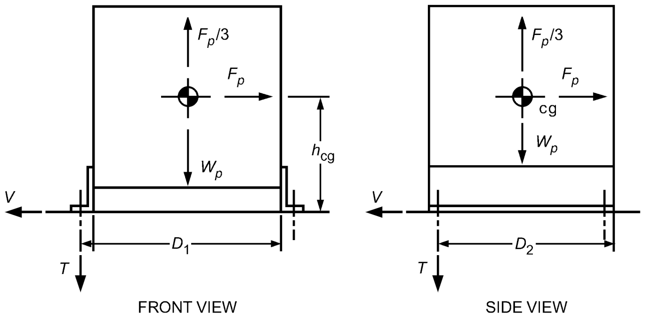


Fig. 1 Equipment with Rigidly Mounted Structural Bases

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

Polar Method

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.

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

The nominal mass component at each restraint is simply the total operating mass divided by the number of restraints. The vertical seismic force is simply the mass component at each location multiplied by the vertical seismic force factor in terms of the total F_{pv} load expressed in *g*s, 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 mass of the equipment). This can be directed either upward or downward when summing forces.

Lump Mass Method

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

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 mass at each restraint point is W_n, then

$$V_{eff} = (F_p/W_p)W_n \tag{7}$$

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.

Resilient Support Factors

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

- For all forces that are not directed along the principal axes, only the corner restraints can be considered to be effective. Thus, for either distribution method, only the corner restraints can be considered capable of absorbing vertical loads.
- If the restraints are independent (separate entities) from the spring isolation elements and if, when exposed to uplift loads, vertical spring forces are not absorbed within the housing of an integral isolator/restraint assembly, the mass factor determined in the first step of the vertical load analysis should be ignored. (This is because any effect that a mass reduction has on the attachment hardware forces is replaced by an approximately equal vertical force component from the spring.)
- If the gap in the restraint element exceeds 6 mm, the final computed forces must be doubled per the IBC.

Building Attachment

The common attachment arrangements are directly bolting with steel bolts and lag bolts, welding, or anchoring to concrete using post-installed anchors. To evaluate the combined effective tensile and shear forces that act simultaneously on these connections, a separate analysis is required.

If allowable stress design (ASD) data are used to size hardware for through-bolted connections for the IBC codes, which are strength based, the loads may be reduced by a factor of 1.4. If LRFD is used when selecting for the hardware, the 1.4 factor does not apply. All allowable capacities used for concrete post-installed anchor bolts selection should be drawn from ICC-ES test reports. These values reflect test data on a single anchor and should be derated for applications where embedment, edge distances, spacing, or location vary from the test conditions. Anchor manufacturers may have selection software to determine anchor bolt capacities that consider installation conditions. It should also be noted that the values published in ES reports may be either ASD or LRFD values and may need to be converted for compatibility with the (LRFD) IBC code being used.

1.5 ANSI STEEL BOLTS

For direct attachment with through bolts using ASD criteria, 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

$$T_{allow} = 179 - 1.8S_v \tag{8}$$

where S_v is the shear stress in the bolt in megapascals. T_{allow}, the maximum allowable tensile stress, must not exceed 138 MPa.

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

1.6 LAG SCREWS INTO TIMBER

Acceptable loads for lag screws into timber can be obtained from the *National Design Specification*® (NDS®) *for Wood Construction* (AWC 2005). 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/embedment depth basis. Note that the values published in this table are capacities in both ASD and LRFD. 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 (8) does not apply. Instead, per Section 9.3.5 of the NDS, the equation is

$$Z_a' = (W'p)Z'/[(W'p)\cos^2\alpha + Z'\sin^2\alpha] \tag{9}$$

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

1.7 CONCRETE POST-INSTALLED ANCHOR BOLTS

Capacities are manufacturer/anchor-type specific. Capacity data should be obtained from the anchor's current ES report. Where failure of the steel does not govern the tensile load, strength-based design (LRFD) should be used. Obtain anchor information from the anchor ICC-ES (formerly ICBO-ESR) report based on anchor and installation factors. For groups of anchors, special factors are required and American Concrete Institute *Standard* 318-08 should be consulted.

ASD Applications

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

$$(T_{eff}/T_{allow\,ASD})^{5/3} + (V_{eff}/V_{allow\,ASD})^{5/3} \le 1.0 \tag{10}$$

or

$$(T_{eff}/T_{allow\,ASD}) + (V_{eff}/V_{allow\,ASD}) \le 1.2 \tag{11}$$

However, if $T_{eff} \le 0.2$; $T_{allow\,ASD}$ the full T_{eff} can equal $T_{allow\,ASD}$, or if $V_{eff} \le 0.2$; $V_{allow\,ASD}$ the full V_{eff} can equal $V_{allow\,ASD}$.

LRFD Applications

The engineer must select an anchor for use from a current evaluation report for anchors that satisfy provisions of ACI 318 Appendix D or ACI 355 (the provisions are the same). From ACI 318, the capacity of the anchor must be reduced in accordance with the following:

$$T = 0.75\phi N \tag{12}$$

$$V = 0.75\phi V \tag{13}$$

The interaction equation for LFRD is modified as follows:

$$(T_{eff}/T_{allow}) + (V_{eff}/V_{allow}) \le 1.2 \tag{14}$$

Types of Concrete Post-Installed Anchors

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.

Undercut anchors expand 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 heavily affected by the skill of the craft and usually not rated for seismic applications.

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. These anchors are usually not rated for seismic applications.

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

Large screw anchors are one-piece anchors that have a concrete cutting thread. These anchors were initially designed to be installed without a specified torque, but torque is used to ensure contact at the rated embedment

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 590°C 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. ES reports have further information on allowable forces for design. Use a safety factor of 2 or as required by ES reports if the installation has not been inspected as required by the IBC Chapter 17 on special inspection.

Stainless steel anchors are required for use in outdoor applications noted in the latest versions of the IBC code.

1.8 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 110 MPa on the throat section of the weld. The section length is 0.707 times the specified weld size.

For a 1.5 mm weld, the length of shear in the weld is 0.707 × 1.5 mm. The allowable weld force $(F_w)_{allow}$ for a 1.5 mm weld is

$$(F_w)_{allow} = 1.06 \times 110 = 117 \text{ N per millimetre of weld} \tag{15}$$

For a 3 mm weld, the capacity is 233 N.

The effective weld force is the sum of the vectors calculated in terms of effective shear and tension shall be reduced. Because the vectors are perpendicular, they are added by the method of the square root of the sum of the squares (SRSS), or

$$(F_w)_{eff} = \sqrt{(T_{eff})^2 \quad (V_{eff})^2} \tag{16}$$

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

$$\text{Weld length} = (F_w)_{eff}/(F_w)_{allow} \tag{17}$$

1.9 SEISMIC SNUBBERS

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

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

Typical snubbers are classified as Types A through J (Figure 2).

Type A. Snubber built into a resilient mounting. All-directional, molded bridge-bearing quality neoprene element is a minimum of 3.2 mm thick.

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

Type C. Spring isolator with built-in all-directional restraints. Restraints have molded neoprene elements with a minimum thickness of 3.2 mm. A neoprene sound pad should be installed between the spring and base plate. Sound pads below the base plate are not recommended for seismic installations.

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

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

Type F. All-directional three-axis snubber with neoprene element. The neoprene element of bridge-bearing quality is a minimum of 4.8 mm thick. Snubber must have a minimum of two anchor bolt holes.

Type G. Lateral snubber. Neoprene element is a minimum of 6.4 mm 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 6.4 mm thick. Minimum floor mounting is with two anchor bolts. Must be used with a minimum of eight, two per corner.

Type J. A telescopic snubber for floor-mounted equipment, with a molded neoprene element, designed to distribute the seismic

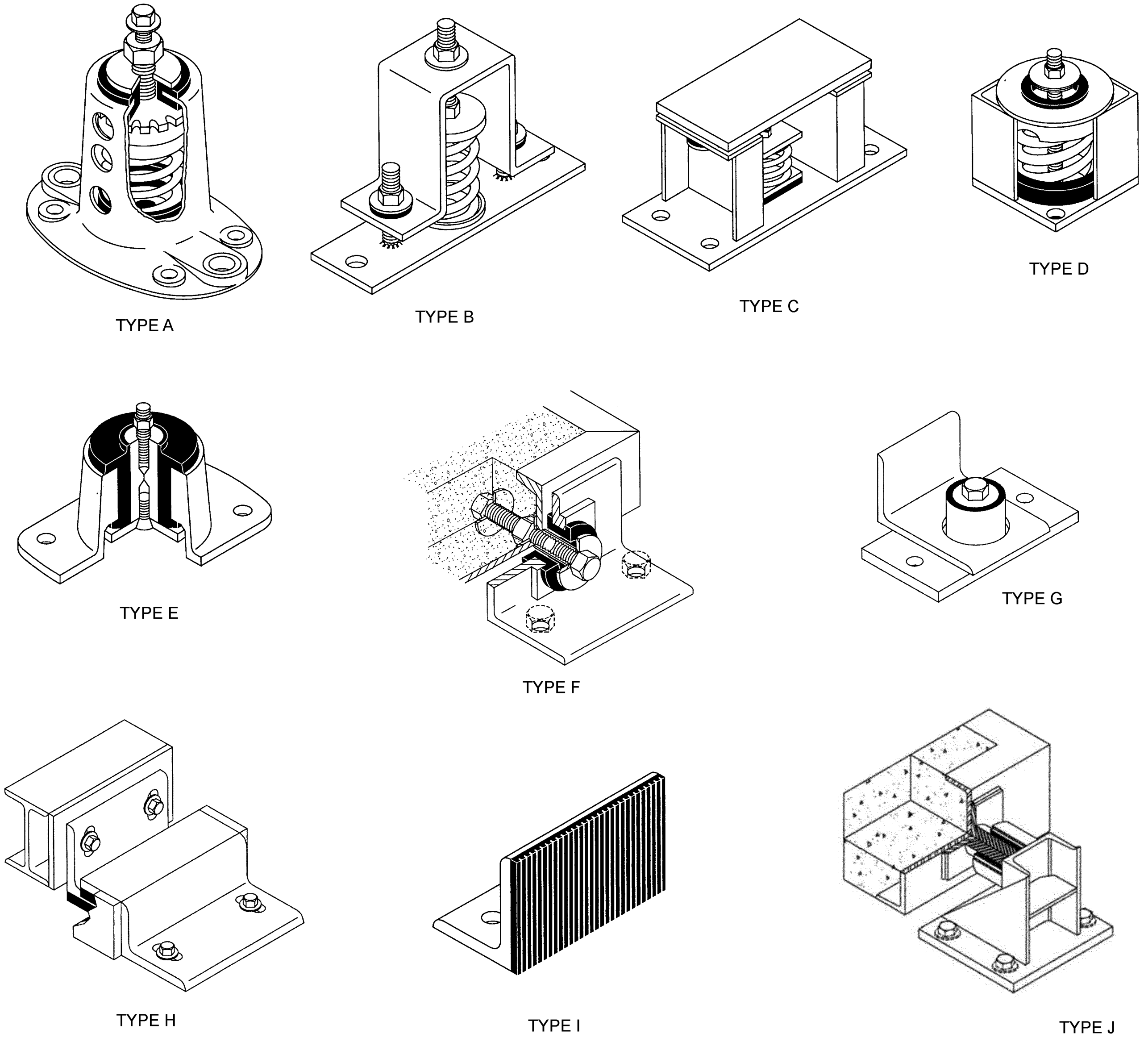

TYPE A
TYPE B
TYPE C
TYPE D
TYPE E
TYPE F
TYPE G
TYPE H
TYPE I
TYPE J

Fig. 2 Seismic Snubbers

force to a larger surface area in a pipe section. This snubber is an all-directional restraint system when a minimum of four snubbers are installed. The molded neoprene element is a minimum of 6.3 mm thick and is installed with an air gap of 4.8 mm not to exceed 6.3 mm.

1.10 SEISMIC RESTRAINTS

For suspended equipment, pipes, ducts, and raceways, it is necessary to restrain lateral movement resulting from seismic acceleration applied to the component. Unrestrained, suspended equipment and related systems will sway violently back and forth, impacting nearby building material and possibly overstressing the hanger rods. The overstressed rods will eventually break and the equipment crash down. To prevent this swaying, suspended components are restrained by one of two methods: a wire rope system, or a rigid brace using steel struts, angles, or other steel elements.

Wire rope restraints are a restraint assembly for suspended equipment, piping, or ductwork consisting of high-strength, galvanized steel aircraft cable. A typical cable restraint system is shown in Figure 3. Cable should have a certified break strength. Some models are color-coded for easy field verification. Cable must be manufactured to meet or exceed minimum materials and standard requirements. Break strengths must be per ASTM E-8 procedures. A safety factor of 2 may be used when prestretched cable is used with end connections designed to meet the cable break strength. Cables are installed to prevent excessive seismic motion and arranged so they do not engage during normal operation. Equipment suspended with vibration isolators must use cable restraints to avoid degrading the isolation. Rigid type bracing will short out the vibration isolators. To prevent buckling of the hanger rods, add a rod stiffener as shown in Figure 4.

Secure the cable to structure and to the braced component through a bracket or stake eye designed to meet the cable restraint rated capacity. Cables are typically secured using one of the following methods:

- Factory-installed permanent stake eye
- Field-looped through bracket and secured with cable grips
- Field-looped through bracket and secured with oval sleeve
- Factory brackets with integral cable clamps to secure cable

When cables are looped through a field-supplied bracket or support hole, a cable thimble should be used to protect the cable. Many factory-supplied brackets are designed specifically for allowing a looped cable through without a thimble. See the manufacturer's instructions for details. Figure 5 shows typical cable restraint details with various attachment methods. Typical attachment methods to secure the rigid brace to the structure and component are shown in Figure 6.

1.11 RESTRAINT OF PIPE AND DUCT RISERS

When piping and ductwork run vertically through a structure, they are identified as risers. They are subject to the same seismic and (less commonly) wind forces as are piping and ductwork oriented horizontally. The primary difference is that the forces that act along the axis of the riser are the summation of the vertical seismic forces and gravity loads, whereas on horizontal systems, the axial forces are simply the horizontal seismic or wind force.

It is also important to recognize, when providing restraint, that risers of any significant length and variation in temperature require support that allows thermally driven changes in the riser's overall length to be accommodated. Because the vertical seismic and wind forces are small compared to gravity forces, axial restraint for the riser can normally be provided with only minor increases in the size of the specialized components used to support the system. Because of the potential of damage to the restraint or support systems as the system grows or shrinks, it is not recommended that redundant axial restraint systems be fitted to a riser. Instead, the primary support system should be designed or selected to meet the job requirement.

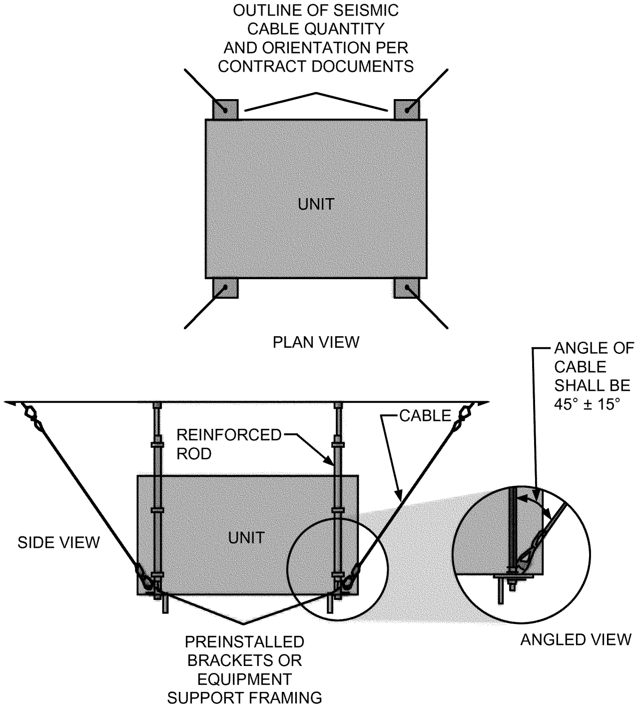


Fig. 3 Cable Restraint

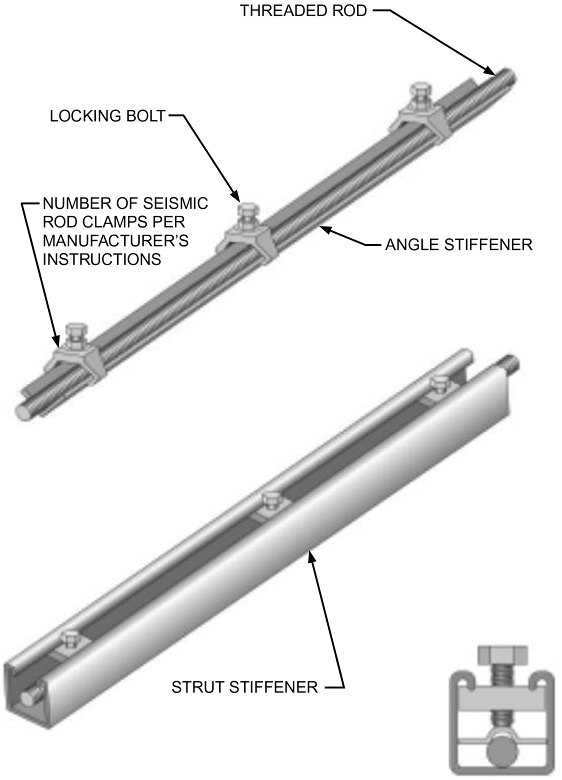


Fig. 4 Rod Stiffener

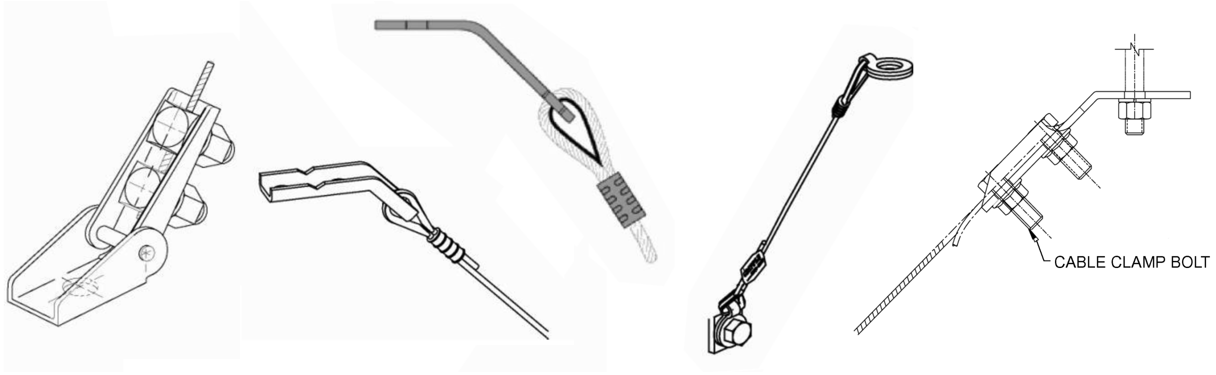


Fig. 5 Types of Cable Connections

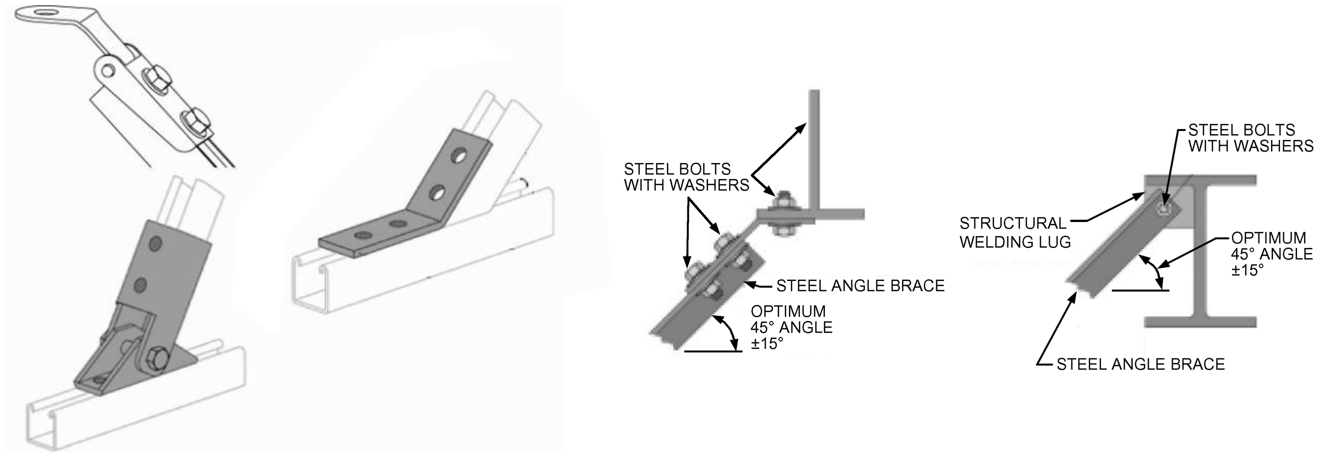


Fig. 6 Strut End Connections

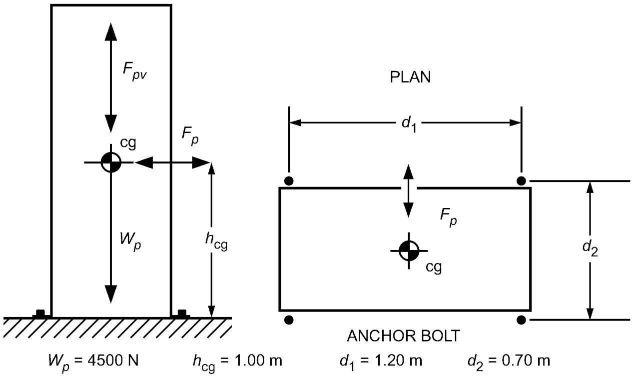


Fig. 7 Equipment Rigidly Mounted to Structure (Example 1)

Risers of significant length are also fitted with some type of stabilization devices. These can be as simple as snug-fitting holes in the floors that the risers penetrate, to specialized brackets or guiding devices that maintain the alignment of the piping or duct while still allowing it to expand or contract. As is the case with the vertical forces, the components used for guidance can frequently be used to provide resistance against seismic or wind events if they are sized and attached appropriately.

If, in the lateral load case, the components used to provide guidance are not adequate to resist the design seismic or wind load conditions, redundant, seismically qualified systems should be fitted to perform this task.

All axial and lateral restraints fitted to risers must be effective against forces that may act in any horizontal or vertical direction as applicable. In addition, the attachment hardware used must be seismically qualified components (e.g., anchors), installed in accordance with seismically qualified procedures.

1.12 EXAMPLES

The following examples are provided to assist in the design of equipment anchorage to resist seismic forces. For Examples 1 through 4, assume the provisions contained in ASCE 7-05 apply, $I_p = 1.5$, $S_s = 0.85$, site soil class is C, and the equipment is located at the top of a 50 m building. Also include an uplift force component $F_{pv} = 0.2 S_{DS} D$ 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.

Note: These examples assume that $I_p = 1.5$. This assumes that the equipment being considered is essential to the continued function of the building following an earthquake or contains hazardous materials. ASCE 7-05 Section 13.2.2 requires that this equipment be certified as being operable after the design earthquake.

Example 1. Anchorage design for equipment rigidly mounted to the structure (see Figure 7).

From Equations (1) to (4), calculate the lateral seismic force and its vertical component. Note that for post-installed, if the anchor satisfies the requirements of ASCE 7, Section 13.4.2, the value of R_p is the same as the component being considered. Post-installed anchors with current evaluation reports published by ICC or other agencies are deemed to be compliant with the provisions of ACI 355 or 318 Appendix D and thus satisfy Section 13.4.2. For rigidly mounted mechanical equipment (period < 0.06 s or > 16.7 Hz), a_p from Table 2 is 1.0, otherwise a_p = 2.5.

ASCE 7 is based on load and resistance factor design (LRFD). In the past, building codes have been based on allowable stress design (ASD). Load factors and load combinations defined in Chapter 2 of ASCE 7 must be considered in design. If a component is anchored with post-installed anchors, the design can only be accomplished using LRFD.

The first step in the load determination process is to determine S_{DS} using the following equation and $F_a = 1.1$ (from Table 3, 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

$$F_P = \left[\frac{0.4 \times 1.0 \times 0.623 \times 4500}{\frac{2.5}{1.5}}\right]\left(1 \quad 2 \times \frac{15}{15}\right) = 2020 \text{ N}$$

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

$$1.6 \times 0.623 \times 1.5 \times 4500 = 6728 \text{ N}$$

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

$$0.3 \times 0.623 \times 1.5 \times 4500 = 1262 \text{ N}$$

Therefore F_p = 2020 N.

When considering provisions of LRFD, a vertical acceleration component must be considered per ASCE 7, Section 12.4.2.2.

$$F_{PV} = 0.2 \times S_{DS} \times D = 0.2 \times 0.623 \times 4500 = 561 \text{ N}$$

For Allowable Stress Design (ASD)

The load combinations of Section 2.4.1 of ASCE 7 must be considered in the design. For rigidly mounted components, Combination 8 is generally the critical combination to be considered.

Calculate the overturning moment OTM:

$$\text{OTM} = F_P h_{cg} = 2020 \times 1.0 = 2020 \text{ N·m} \qquad (18)$$

Calculate the resisting moment RM:

$$\mathrm{RM} = W_P\left(\frac{d_{min}}{2}\right) = 4500\left(\frac{70}{2}\right) = 1575\ \mathrm{N\cdot m} \tag{19}$$

$$T = [2020(0.7) - 1575(0.6)]/0.70 = 670\ \mathrm{N} \tag{20}$$

Calculate T_{eff} per bolt:

$$T_{eff} = 670/2 = 335\ \mathrm{N}\ \text{per through-bolt or lag screw} \tag{21}$$

Calculate shear force per bolt:

$$V_{eff} = 2020/(4 \times 1.4) = 361\ \text{lb per through-bolt or lag screw} \tag{22}$$

Load and Resistance Factor Design (LRFD)

The load combinations of Section 2.3.2 of ASCE 7 must be considered in the design. For rigidly mounted components, Combination 7 is generally the critical combination to be considered.

$$\mathrm{RM} = (W_p - F_{pv})d_{min}/2 = (4500 - 561)0.70/2 = 1379\ \mathrm{N\cdot m} \tag{23}$$

$$T_{eff} = [2020 - 0.9(1379)]/0.70(2) = 556\ \mathrm{N} \tag{24}$$

Per ASCE 7 Section 13.4.2, if post-installed anchors are used, an additional 1.3 factor is applied to all E loads. The 1.3 factor applies for projects using ASCE 7-05. This factor is not required for applications using ASCE 7-10.

$$\mathrm{OTM} = 2020(1.3) = 2626\ \mathrm{N\cdot m} \tag{25}$$

$$T_{eff} = [2626 - 1379(0.9)]/0.70(2) = 989\ \mathrm{N} \tag{26}$$

To convert in·lb to N·m, multiply by 0.113.

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 361/1.4 or 258 N per bolt and the pullout load T_{eff} becomes 335/1.4 = 239 N 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

$$T\alpha_{eff} = \sqrt{(T_{eff})^2 + (V_{eff})^2} = \sqrt{(258)^2 \quad (239)^2} = 352\ \mathrm{N}$$

The angle α = arcs in $(T_{eff}/Z'_{\alpha}) = 42.5°$, where $Z\alpha$ is the allowable lag screw load multiplied by applicable factors and $Z'_{\alpha}\alpha$ 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 13 mm diameter, 100 mm 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 2277 N.

From Table 9.2A in the NDS, for $G = 0.37$ and 90 mm full thread, $W = 66.7 \times 90 = 6003$ N.

Substituting into the combined load for lag bolts [Equation (9)] gives

$$Z'_{\alpha} = \frac{(66.7 \times 90)2277}{(66.7 \times 90)42.5 \quad 2277\sin^2 42.5} = 3177\ \mathrm{N}$$

Therefore, a 13 mm diameter, 100 mm long lag screw can be used at each corner of the equipment.

Case 2. *Equipment attached to steel*

For equipment attached directly to a steel member, analysis is the same as that shown in case 1. Capacities for the attaching bolts are given in the *Manual of Steel Construction* (AISC 1989). See Chapter J of the AISC Specification for design provisions.

For this example $T_{eff}/T_{ASD} = 556/4410 = 0.126 < 0.2$; therefore a combined tension shear check need not be performed on the connection.

Therefore, 13 mm diameter bolts can be used.

Example 2. Anchorage design for equipment supported by external spring mounts (Figure 8) 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 ASCE 7, 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 2 is 2.5.

The basic force equation is then (*Note*: using $R_p = 1.5$ is conservative, because the anchors must comply with 13.4.2 of ASCE 7. The numbers could be modified)

$$F_P = \left(\frac{0.4 \times 2.5 \times 0.623 \times 4500}{\frac{1.5}{1.5}}\right)\left(1 \quad 2 \times \frac{50}{50}\right) = 8411\ \mathrm{N}$$

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

$$1.6 \times 0.623 \times 1.5 \times 4500 = 6728\ \mathrm{N}$$

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

$$0.3 \times 0.623 \times 1.5 \times 4500 = 1262\ \mathrm{N}$$

The vertical force F_{pv} equals

$$F_{PV} = 0.2S_{DS}D = 0.2 \times 0.623 \times 4500 = 561\ \mathrm{N}$$

Because the equipment is resiliently supported, footnote b to Table 13.6-1 of ASCE 7 indicates that the computed forces may need to be doubled. Therefore, F_p is 6728 × 2 = 13.5 kN and F_{pv} is 561 × 2 = 1.1 kN.

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 and C = maximum compression on isolator, then

$$\begin{aligned} T, C &= \frac{-W_P + F_{PV}}{4} \pm F_P h_{cg}\frac{\cos\theta}{2b} + F_P h_{cg}\frac{\sin\theta}{2a} \\ &= \frac{-W_P + F_{PV}}{4} \pm \frac{F_P h_{cg}}{2}\left(\frac{\cos\theta}{b} + \frac{\sin\theta}{a}\right) \end{aligned} \tag{27}$$

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

$$\frac{dT}{d\theta} = \frac{F_P h_{cg}}{2}\left(\frac{\cos\theta}{b} + \frac{\sin\theta}{a}\right) = 0 \tag{28}$$

$$\theta_{max} = \tan^{-1}(b/a) = \tan^{-1}(0.7/1.2) = 30.26° \tag{29}$$

$$T = \frac{-W_P + F_{PV}}{4} + \frac{F_P h_{cg}}{2}\left(\frac{\cos\theta_{max}}{b} + \frac{\sin\theta_{max}}{a}\right) \tag{30}$$

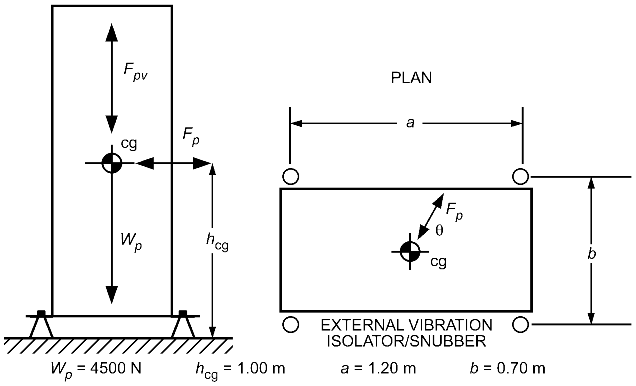


Fig. 8 Equipment Supported by External Spring Mounts

$$C = \frac{-W_P + F_{PV}}{4} - \frac{F_P h_{cg}}{2}\left(\frac{\cos\theta_{max}}{b} + \frac{\sin\theta_{max}}{a}\right) \tag{31}$$

$$T = \frac{-4.5 \quad 1.12}{4} \quad \frac{13.5 \times 1}{2}\left(\frac{\cos 30.26}{0.7} + \frac{\sin 30.26}{1.2}\right) = 10.3 \text{ kN}$$

$$T = \frac{-4.5 \quad 1.12}{4} \quad \frac{13.5 \times 1}{2}\left(\frac{\cos 30.26}{0.7} + \frac{\sin 30.26}{1.2}\right) = -12.0 \text{ kN}$$

Calculate the shear force per isolator:

$$V = (F_P/N_{iso}) = 13\,500/4 = 3375 \text{ N} \tag{32}$$

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.

$$(T_1)_{eff} \text{ per bolt} = T/2 = 10\,300/2 = 5150 \text{ N} \tag{33}$$

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

$$(T_2)_{eff} = V \times \text{operating height}/dN_{bolt} \tag{34}$$

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

$$(T_2)_{eff} = (3375 \times 8)/(3 \times 2) = 4500 \text{ N} \tag{35}$$

$$(T_{max})_{eff} = (T_1)_{eff} + (T_2)_{eff} = 5150 + 4500 = 9650 \text{ N} \tag{36}$$

$$V_{eff} = 3375/2 = 1688 \text{ N} \tag{37}$$

See Example 1 for the design of the connections to the structural system.

Example 3. Anchorage design for equipment with a center of gravity different from that of the isolator group (Figure 10).

Anchor properties

$$I_x = 4B^2 \qquad I_y = 4L^2 \tag{38}$$

Angles:

$$\theta = \tan^{-1}(B/L) \tag{39}$$

$$\alpha = \tan^{-1}(e_x/e_y) \tag{40}$$

$$\beta = 180\ |\alpha - \theta\ | \tag{41}$$

$$\phi = \tan^{-1}(LI_x/BI_y) \tag{42}$$

Vertical reactions

$$(W_n)_{max/min} = W_p \pm F_{pv} \tag{43}$$

Vertical reaction caused by overturning moment

$$T_m = \pm F_P\left(\frac{Bh_{cg}}{I_x}\cos\theta + \frac{Lh_{cg}}{I_y}\sin\theta\right) \tag{44}$$

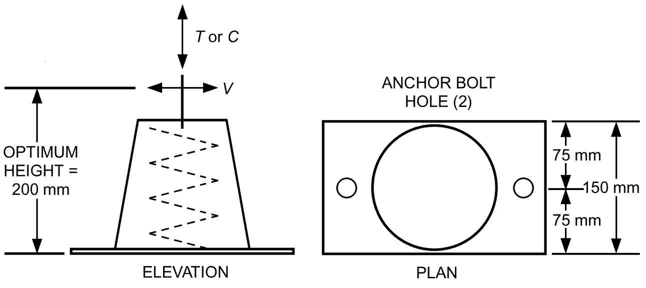


Fig. 9 Spring Mount Detail (Example 2)

Vertical reaction caused by eccentricity

$$(T_e)_{max/min} = (W_n)_{max/min}\left(\frac{Be_y}{I_x} \quad \frac{Le_y}{I_y}\right) \tag{45}$$

Vertical reaction caused by W_p

$$(T_w)_{max/min} = (W_n)_{max/min}/4 \tag{46}$$

$$T_{eff} = T_m + (T_e)_{max} + (T_w)_{max} \text{ (always compression)} \tag{47}$$

$$T_{eff} = -T_m + (T_e)_{min} + (T_w)_{min} \text{ (tension if negative)} \tag{48}$$

Horizontal reactions

Horizontal reaction caused by rotation

$$V_{rot} = F_P\left(\frac{e_x^2 \quad e_y^2}{16(B^2 \quad L^2)}\right)^{0.5} \tag{49}$$

$$V_{dir} = F_p/4 \tag{50}$$

$$V_{max} = (V_{rot}^2 + V_{dir}^2 - 2V_{rot}V_{dir}\cos\beta)^{0.5} \tag{51}$$

See Example 1 for the design of the connections to the structural system.

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

Example 4. Anchorage design for equipment with supports and bracing for suspended equipment (Figure 11). Equipment mass $W_p = 2200$ N.

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.

Anchor properties

$$I_x = 4B^2 \qquad I_y = 4L^2 \tag{52}$$

Angle

$$\phi = \tan^{-1}(LI_x/BI_y) = 36.86° \tag{53}$$

From Equation (43),

$$(W_n)_{max/min} = 2200 \pm 548 = 2748 \text{ N or } 1652 \text{ N}$$

From Equation (44),

$$T_m = \pm 4934(0.132 + 0.075) = \pm 1021 \text{ N}$$

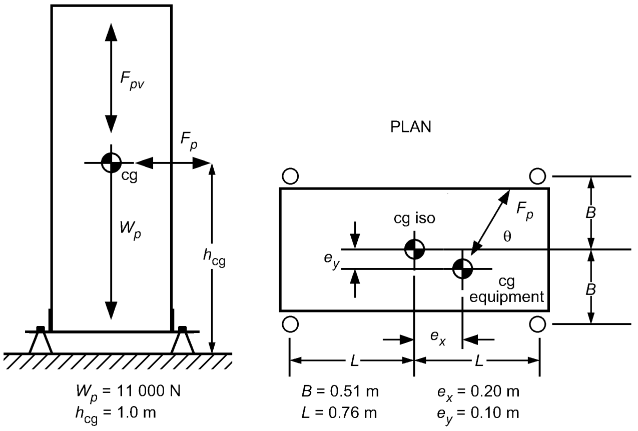


Fig. 10 Equipment with Center of Gravity Different from Isolator Group (in Plan View)

From Equation (45),

$$T_e = 0$$

From Equation (46),

$$(T_w)_{max/min} = 687 \text{ N or } 413 \text{ N}$$

From Equation (47),

$$(T_{eff})_{max} = 1021 + 0 + 687 = 1708 \text{ N (downward)}$$

From Equation (48),

$$(T_{eff})_{max} = -1021 + 0 + 413 = -608 \text{ N (upward)}$$

Forces in the hanger rods:

Maximum tensile = 1708 N

Maximum compression = 608 N

Force in the splay brace = $F_P\sqrt{2}$ = 6978 N at a 1:1 slope.

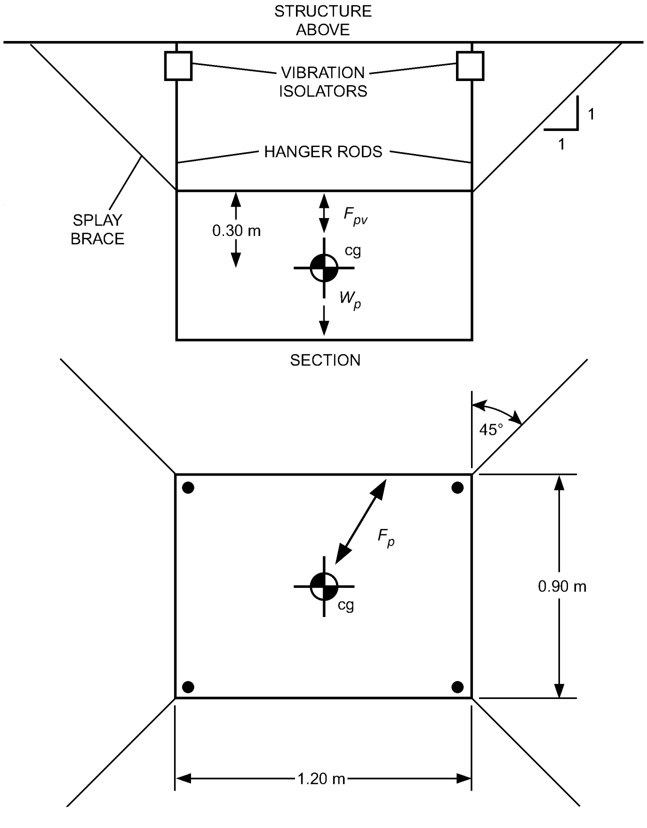
Because of the force being applied at the critical angle, as in Example 2, only one splay brace is effective in resisting the lateral load F_p.

Design of hanger rod/vibration isolator and connection to structure

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

Determine whether a 13 mm wedge anchor with special inspection provisions will hold the required load.

$$T_{allow} = 2600 \times 0.5 \times 2 = 2600 > T_{eff} = 1708 \text{ N}$$



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

Fig. 11 Supports and Bracing for Suspended Equipment

Therefore, a 13 mm rod and post drill-in anchor should be used at each corner of the unit.

For anchors installed without special inspection,

$$T_{allow} = 2600 \times 0.5 = 1300 > T_{eff} = 1708 \text{ N}$$

Therefore, a larger anchor should be chosen.

Determine if the 13 mm hanger rod would require a stiffener if it is 1 m long.

Design of splay brace and connection to structure

Force in the slack cable = 6978 N.

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

$$V_{max} = 4934 \text{ N } T_{max} = F_p = 4934 \text{ N}$$

Because the cable forces are relatively small, a 9.5 mm 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.

The design of a post-installed anchor installation is similar to that shown in Example 1. Anchors installed through a metal deck will have lower capacities than anchors installed in a flat slab because of limited embedment depths. Take care to ensure that the design also satisfies the requirements contained in the evaluation report for the anchor specified.

Prescriptive provisions of ASCE 7 can be summarized as follows:

- Formulas for relative displacement of floor and ceiling can be conservatively estimated at 1% of the floor-to-ceiling height. This displacement must be used to determine the required horizontal flexibility of the pipe, duct, or electrical connections at the equipment interface.
- In ASCE 7, using all-directional snubbers with clearance of more than 7 mm increases F_p by a factor of 2.
- Component supports must be designed to accommodate component movement to prevent pounding on the structure or other components. This affects internal isolators and snubbers.
- Equipment components exposed to seismic impact forces and using nonductile housings must be designed using 25% of material yield stresses.
- Nonessential equipment, failure of which can cause essential equipment failure, must be designed as essential equipment.
- If the structure's site class is not provided in the contract documents, assume site class D, subject to change by the building official.
- For pipe or duct on any given run, if the distance from the bottom of the structure to the top of the support is 305 mm or less for all supports in that run, then that run does not need sway braces.
- Pipe and ducts may not be required to have sway braces, depending on size, material content, and importance factor. These conditions are defined in Chapter 13 of ASCE 7.

1.13 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. The evaluation report for the chosen anchor should be followed as a guide for edge distances and center-to-center spacing.
- Supplementary steel bases and frames, concrete bases, or equipment modifications may void some manufacturers' warranties. Snubbers, for example, should be properly attached to a subbase. Bumpers may be used with springs.
- Static analysis does not account for the effects of resonant conditions within a piece of equipment or its components. Because all equipment has different resonant frequencies during operation and nonoperation, the equipment itself might fail even if the restraints do not. Equipment mounted inside a housing should be

seismically restrained to meet the same criteria as the exterior restraints.

- Snubbers used with spring mounts should withstand motion in all directions. Some snubbers are only designed for restraint in one direction; sets of snubbers or snubbers designed for multidirectional purposes should be used.
- Equipment must be strong enough to withstand the high deceleration forces developed by resilient restraints.
- Flexible connections should be provided between equipment that is braced and piping and ductwork that need not be braced.
- Flexible connections should be provided between isolated equipment and braced piping and ductwork.
- Bumpers installed to limit horizontal motion should be outfitted with resilient neoprene pads to soften the potential impact loads of the equipment.
- Anchor installations must be inspected (usually required for anchors resisting seismic forces); in many cases, damage occurs because bolts were not properly installed. To develop the rated restraint, bolts should be installed according to manufacturer's recommendations.
- Brackets in structural steel attachments should be matched to reduce bending and internal stresses at the joint.
- With the exception of heavy-duty clamps used to attach longitudinal restraints to piping systems, friction must not be relied on to resist any load. All connections should be positive and all holes should be tight-fitting or grouted to ensure minimal clearance at the attachment points.

2. WIND-RESISTANT 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. Two main areas of the HVAC&R system are exposed to wind events: the HVAC&R equipment and the exterior wall-mounted cladding components, such as intake and exhaust louvers. For HVAC&R equipment, 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. For wall-mounted components, a design wind pressure P is determined, which can be used to specify equipment performance levels and design anchors to adequately brace wall-mounted cladding components to the building structure.

The American Society of Civil Engineers' (ASCE) *Standard* 7-05 includes design guidelines for wind, snow, rain, and earthquake loads. Note that the equations, guidelines, and data presented here only cover nonstructural components. The current standard (2005) includes more comprehensive and rigorous procedures for evaluating wind forces and wind restraint. Refer to the latest version of ASCE *Standard* 7 adopted by the local jurisdiction.

2.1 TERMINOLOGY

Classification. Buildings and other structures are classified for wind load design exposure according to Table 7.

Basic wind speed. The fastest m/s wind speed at 10 m above the ground of Terrain Exposure C (see Table 7) having an annual probability of occurrence of 0.02. Data in ASCE *Standard* 7 or regional climatic data may be used to determine basic wind speeds. ASCE data do not include all special wind regions (such as mountainous terrains, gorges, and ocean promontories) where records or experience indicate that the wind speeds are higher than what is shown in appropriate wind data tables. For these circumstances, regional climatic data may be used provided that both acceptable extreme-value statistical analysis procedures were used in reducing the data and that due regard was given to the length of record, averaging time, anemometer height, data quality, and terrain exposure. One final exclusion is that tornadoes were not considered in developing the basic wind speed distributions.

Components and Cladding. Elements of the building envelope that do not qualify as part of the main wind-force resisting system.

Corner Zone. Areas of building walls and roofs adjacent to building corners that experience increased external pressure from wind.

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

Importance factor *I*. A factor that accounts for the degree of hazard to human life and damage to HVAC components (Table 8). For hurricanes, the value of the importance factor can be linearly interpolated between the ocean line and 160 km inland because wind effects are assumed negligible at this distance inland.

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

Minimum design wind load. The wind load may not be less than 0.48 kPa multiplied by the area of the HVAC component projected on a vertical plane that is normal to the wind direction.

2.2 CALCULATIONS

Two procedures are used to determine the design wind load on HVAC components. The **analytical procedure**, described here, is the most common method for standard component shapes, based on the requirements in ASCE 7. The second method, the **wind-tunnel procedure**, is used in the analysis of complex and unusually shaped components or equipment located on sites that produce wind

Table 7 Definition of Exposure Categories

Exposure B. Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions the size of single-family dwellings or larger. Use of this exposure category is limited to those areas for which terrain representative of Exposure B prevails upwind for at least 800 m or 20 times the height of the building or structure, whichever is greater.

Exception: For buildings with mean roof height less than or equal to 9 m, the upwind distance may be reduced to 460 m.

Exposure C. Open terrain with scattered obstructions having heights generally less than 9 m. This category includes flat open country, grasslands, and all water surfaces in hurricane-prone areas. Exposure C applies for all cases where Exposure B or D do not apply.

Exposure D. Flat, unobstructed areas exposed to wind and flowing over open water outside of hurricane-prone regions for a distance of at least 1600 m. This exposure applies to structures exposed to the wind coming from over the water as well as smooth mud flats, salt flats, and unbroken ice.

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Notes:

1. For a site located in a transition zone between exposure categories, the exposure resulting in the largest wind forces must be used.
2. Exposure Category D extends into downwind areas of Exposures B or C for a distance of 200 m or 20 times the height of the building, whichever is greater.
3. The responsibility for determining the exposure category for a given new building project falls on the structural engineer of record. This value is documented in the structural notes drawing (the first of the structural drawings) for the project.

Table 8 Wind Importance Factor *I* (Wind Loads)

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

Note: See Table 9 for categories.

Table 9 Exposure Category Constants

Exposure Category	α	Z_g, m	Gust Factor *G*
B	7	360	0.85
C	9.5	270	0.85
D	11.5	210	0.85

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Note: See Table 7 for definitions of exposure categories.

Table 10 Force Coefficients for HVAC Components, Tanks, and Similar Structures

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

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

channeling or buffeting because of upwind obstructions. The analytical procedure produces design wind forces that are expected to act on HVAC components for durations of 1 to 10 s. The various factors, pressure, and force coefficients incorporated in this procedure are based on a mean wind speed that corresponds to the fastest wind speed.

Analytical Procedure

The design wind force is determined by the following equation:

$$F_w = Q_z G C_f A_F \tag{54}$$

where

F_w = design wind force, N
Q_z = velocity pressure evaluated at height *z* above ground level, Pa
G = gust response factor for HVAC components evaluated at height *z* above ground level
C_f = force coefficient (Table 10)
A_f = area of HVAC component projected on a plane normal to wind direction, m^2

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

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

$$Q_z = 0.613 K_z K_{zt} K_d V^2 I \tag{55}$$

where

K_z = velocity pressure exposure coefficient from Table 12
K_{zt} = topographic factor = 1.0
K_d = wind directionality factor = 1.0
V = velocity from Figure 12, m/s
I = importance factor from Table 8

The force generated by the wind is calculated by

$$F_w = Q_z G C_f A_f \tag{56}$$

where

F_w = design wind force, N
Q_z = velocity pressure evaluated at height *z* above ground level, Pa
G = gust response factor for HVAC components evaluated at height *z* above ground level
C_f = force coefficient (Table 10)
A_f = area of HVAC component projected on a plane normal to wind direction, m^2

The following example calculations are for a 1400 kW cooling tower:

Tower height $h = 3$ m
Tower width $D = 3$ m
Tower length $l = 6$ m
Tower operating mass $W_p = 8650$ kg
Tower diagonal dimension = $\sqrt{3^2 \quad 6^2} = 6.71$ m
Area normal to wind direction $A_f = 3 \times 6.71 = 20.1$ m^2

From Table 10, $C_f = 1.0$ for wind acting along diagonal with $h/D = 3/3 = 1$.

Example 5. Suburban hospital in Omaha, Nebraska. The top of the cooling tower is 30 m above ground level. Building width normal to the wind $B = 1000$ m, and building height $H = 35$ m.

Solution:

From Figure 12, the design wind speed is found to be 40 m/s.
From Table 9, use Category IV.
From Table 7, use Exposure B.
From Table 8, $I = 1.15$.
From Table 12, $K_z = 1.07$.
From Figure 13, $K_d = 0.9$.
From Table 9, $G = 0.85$.

Substitution into Equation (58) yields

$$Q_z = 0.613 \times 1.07 \times 1.0 \times 0.9 \times (40)^2 \times 1.15 = 1086 \text{ Pa} = 1.1 \text{ kPa}$$

Building height is greater than 20 m; therefore, $E_f = 1.0$.
Substitution into Equation (54) yields the design wind force as

$$F_w = 1086 \times 0.85 \times 1.0 \times 20.1 \times 1.0 = 19\,460 \text{ N}$$

Example 6. Office building in New York City. Top of tower is 200 m above ground level. Building wall normal to the wind $B = 180$ m and building height $H = 190$ m.

Solution:

From Figure 12, the design wind speed is 54 m/s.
From Table 9, use Category II.
From Table 7, use Exposure B.
From Table 8, $I = 1.0$.
From Figure 13, $K_d = 0.9$.
Because $z > 150$ m, K_z must be determined from Note 2 of Table 12.
From Table 9, $\alpha = 7.0$, $z_g = 1200$, and $G = 0.85$.

Substituting into the first equation in Note 2 yields

$$K_z = 2.10(Z/Z_g)^{2/\alpha} = 1.72$$

Substituting into Equation (55) yields

$$Q_z = 0.613 \times 1.72 \times 1.0 \times 0.9 \times (54)^2 \times 1.15 = 3182 \text{ Pa}$$

Building height is greater than 20 m, therefore $E_f = 1.0$.

Substituting into Equation (56) yields the design force wind as

$$F_W = 3182 \times 0.85 \times 1.0 \times 20.1 \times 1.0 = 54\,364 \text{ N}$$

Example 7. Church in Key West, Florida. The top of the tower is 15 m above ground level. Building wall normal to the wind $B = 110$ m and building height $H = 12$ m.

Solution:

From Figure 12, the design speed is found to be 67 m/s.
From Table 9, use Category III.
From Table 5, use Exposure C (as this is a hurricane-prone region).
From Table 16, $I = 1.15$.
From Table 17, $G = 0.85$.
From Table 13, $K_d = 0.9$.
From Table 12, $K_z = 1.09$ (for exp category C).
From Equation (55):

$$Q_z = 0.613 \times 1.09 \times 1.0 \times 0.9 \times (67)^2 \times 1.15 = 3104 \text{ Pa}$$

Building height is less than 20 m, $A_f/(B \times H) = 224/(300 \times 40) = 0.02$, therefore, $E_f = 1.9$

Substituting into Equation (56) gives the design wind force as

$$F_w = 3104 \times 0.85 \times 1.0 \times 20.1 \times 1.9 = 100\ 760 \text{ N}$$

2.3 WALL-MOUNTED HVAC&R COMPONENT CALCULATIONS (LOUVERS)

For many projects, the structural engineer of record will determine the components and cladding wind pressures provided on the structural notes drawing. If these wind pressures are not provided, the two following procedures (described previously) are used to determine the design wind load on HVAC cladding components.

Analytical Procedure

Velocity Pressure. The design wind speed must be converted to a velocity pressure that is acting on an HVAC component at height z above the ground. This is done using Equation (54). Once the velocity pressure has been determined, the design wind pressure can be calculated.

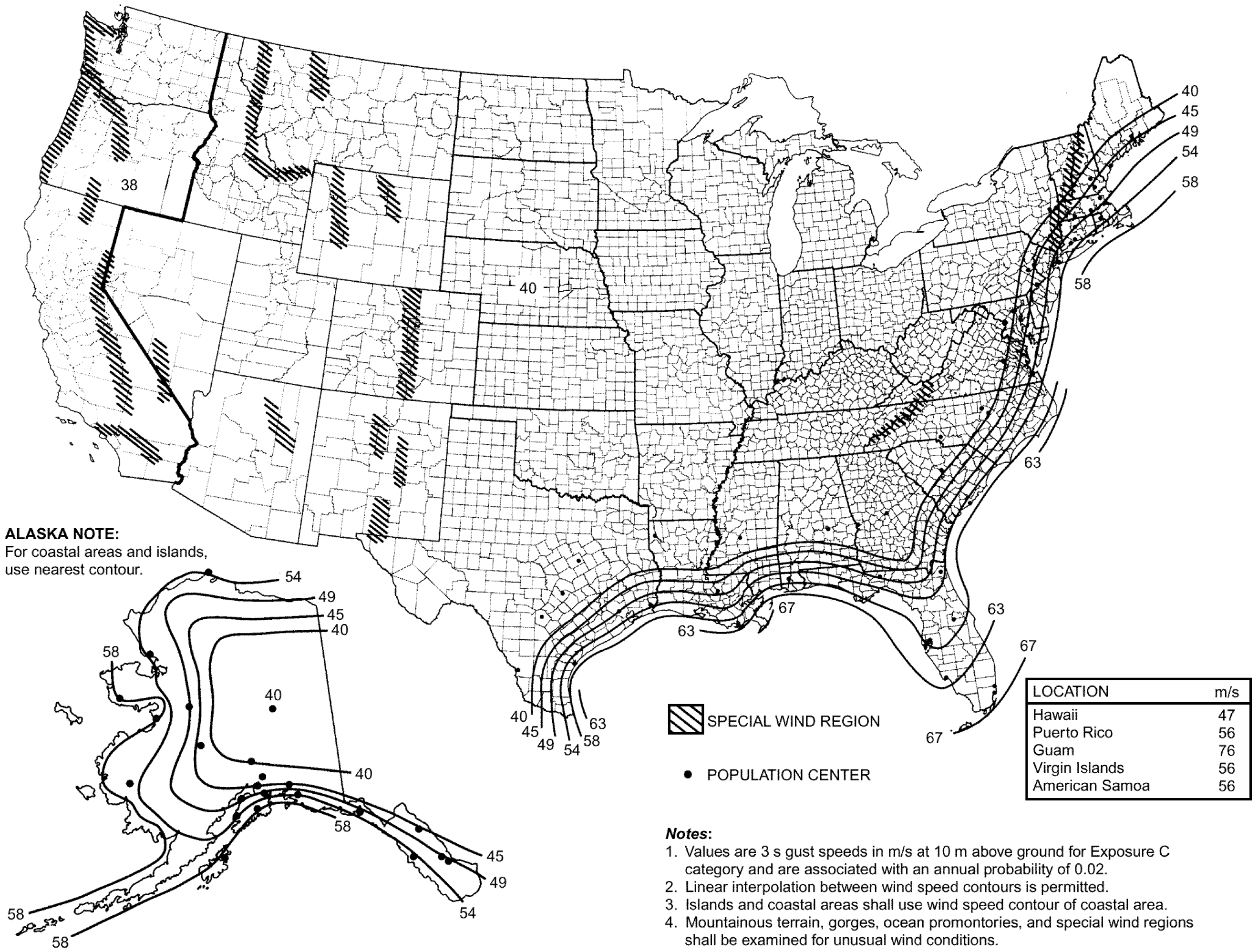
Low-Rise Buildings and Buildings with h $\leq$ *18.3 m*

The design wind pressure for cladding is determined by the following equation:

$$P_w = Q_h(GC_p - GC_{pi}) \tag{57}$$

where

P_w = design wind pressure, N/m²
Q_h = velocity pressure evaluated at mean roof height h above ground level, Pa
GC_p = external pressure coefficient given in Figure 13
GC_{pi} = internal pressure coefficient given in Table 14



ALASKA NOTE:
For coastal areas and islands, use nearest contour.

▨ SPECIAL WIND REGION

● POPULATION CENTER

LOCATION	m/s
Hawaii	47
Puerto Rico	56
Guam	76
Virgin Islands	56
American Samoa	56

***Notes*:**
1. Values are 3 s gust speeds in m/s at 10 m above ground for Exposure C category and are associated with an annual probability of 0.02.
2. Linear interpolation between wind speed contours is permitted.
3. Islands and coastal areas shall use wind speed contour of coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Fig. 12 Wind Speed Data
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Buildings with h *> 18.3 m*

The design wind pressure is determined by the following equation:

$$P_w = Q(GCp) - q_i(GC_{pi}) \qquad (58)$$

where

P_w = design wind pressure, N/m^2
Q_z = velocity pressure for windward walls calculated at height z above the ground of the component being examined
Q_h = velocity pressure for leeward walls, side walls and roofs, evaluated at height h of the roof
Q_i = velocity pressure for windward walls, side walls, leeward walls, and roofs, evaluated at height h of the roof
GC_p = external pressure coefficient given in Figure 14
GC_{pi} = internal pressure coefficient given in Table 14

Example 8. Office building in Houston, Texas. The top of the building is 9.1 m above grade located in a newly developed suburban area. It is necessary to determine the wind pressures on louver 1 and louver 2 shown on the building elevation in Figure 15.

Solution:

From Figure 12, the design speed is found to be 54 m/s.
From Table 9, use Category II.
From Table 7, use Exposure C.
From Table 8, $I = 1.0$.
From Table 12, $K_z = 0.98$, at roof height, $h = 9.1$ m.
From Table 13, $K_d = 0.85$.
K_{zt} assumed to be 1.0.

Determine GC_p: Building height h is less than 18.3 m; therefore, Equation (55) is used for the pressure evaluations. GC_p must be determined from Figure 15 for each of the louvers.

Louver 1: from the notes on Figure 15, it is necessary to determine the a dimension, which establishes the corner zone 5. The least horizontal dimension coming into the corner is 9.8 m from the plan view. Ten percent of this value is 1 m. The minimum value for the corner dimension is 0.9 m. Louver 1 is located 0.37 m from the corner and is therefore in corner zone 5.

From Figure 15, GC_p = +0.95 or –1.3 for a 1.9 m^2 wind area. A positive GC_p indicates a positive pressure on the windward side of the building. A negative GC_p indicates a suction pressure on the leeward side of the building. Both cases must be evaluated.

Louver 2: based on the corner calculation, louver 2 is in noncorner zone 4. From Figure 15, GC_p = +0.9 or –1.0 for a 2.8 m^2 wind area.

Determine GC_{pi}: See Figure 14. Most buildings without significant wall openings are enclosed buildings. For the purposes of this example, an enclosed building is assumed. GC_{pi} = +0.18 or –0.18. A positive sign indicates pressure outward on all structure walls. A negative sign indicates pressure inward on all structure walls.

Determine velocity pressure at roof elevation h from Equation (55):

$$Q_h = 0.613 \times 0.98 \times 1.0 \times 0.85 \times (54)^2 \times 1.0 = 1489 \text{ Pa}$$

Determine design wind pressure P from Equation (56):

Louver 1, case 1: positive external, positive internal

$$P = 1489 \times (0.95 - 0.18) = 1147 \text{ Pa}$$

Louver 1, case 2: positive external, negative internal

$$P = 1489 \times [0.95 - (-0.18)] = 1683 \text{ Pa}$$

Louver 1, case 3: negative external, positive internal

$$P = 1489 \times [(-1.3) - 0.18] = -2204 \text{ Pa}$$

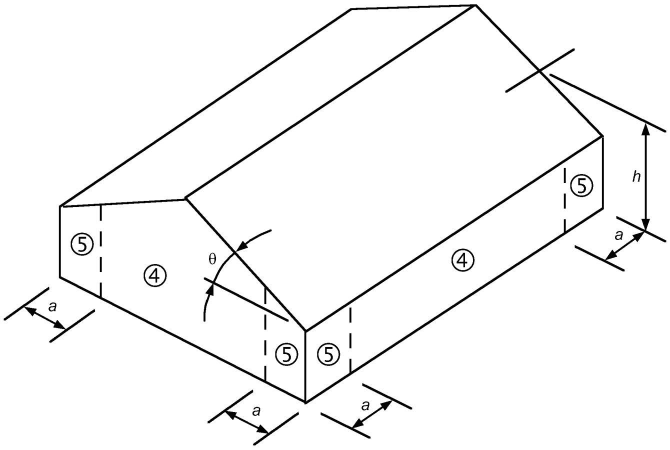
Louver 1, case 4: negative external, negative internal

$$P = 1489 \times [(-1.3) - (-0.18)] = -1667 \text{ Pa}$$

The controlling values for P for louver 1 are 1683 Pa, –2204 Pa and should be used to specify equipment performance levels.

Louver 2, case 1: positive external, positive internal

$$P = 1489 \times (0.90 - 0.18) = 1072 \text{ Pa}$$



NOTES:
1. Vertical scale denotes GC_p to be used with q_h.
2. Horizontal scale denotes effective wind area, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Each component shall be designed for maximum positive and negative pressures.
5. Values of GC_p for walls shall be reduced by 10% when θ ≤ 10°.
6. Notation:
   - *a*: 10% of least horizontal dimension or 0.4*h*, whichever is smaller, but not less than either 4% of least horizontal dimention or 0.9 m.
   - *h*: Mean roof height, in metres, except that eave height shall be used for θ ≤ 10°.
   - θ: Angle of plane of roof from horizontal, in degrees.

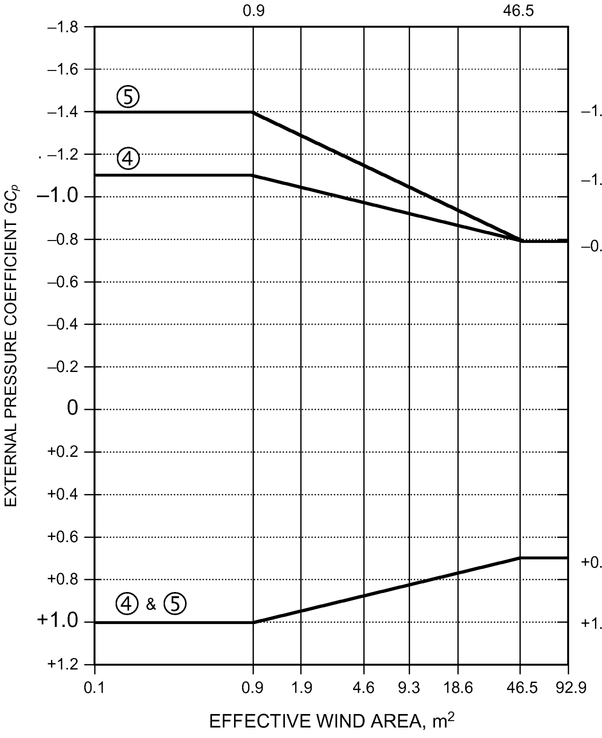


Fig. 13 External Pressure Coefficient GC_p for Walls for $h \le 18.3$ m

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Louver 2, case 2: positive external, negative internal

$$P = 1489 \times [0.90 - (-0.18)] = 1608 \text{ Pa}$$

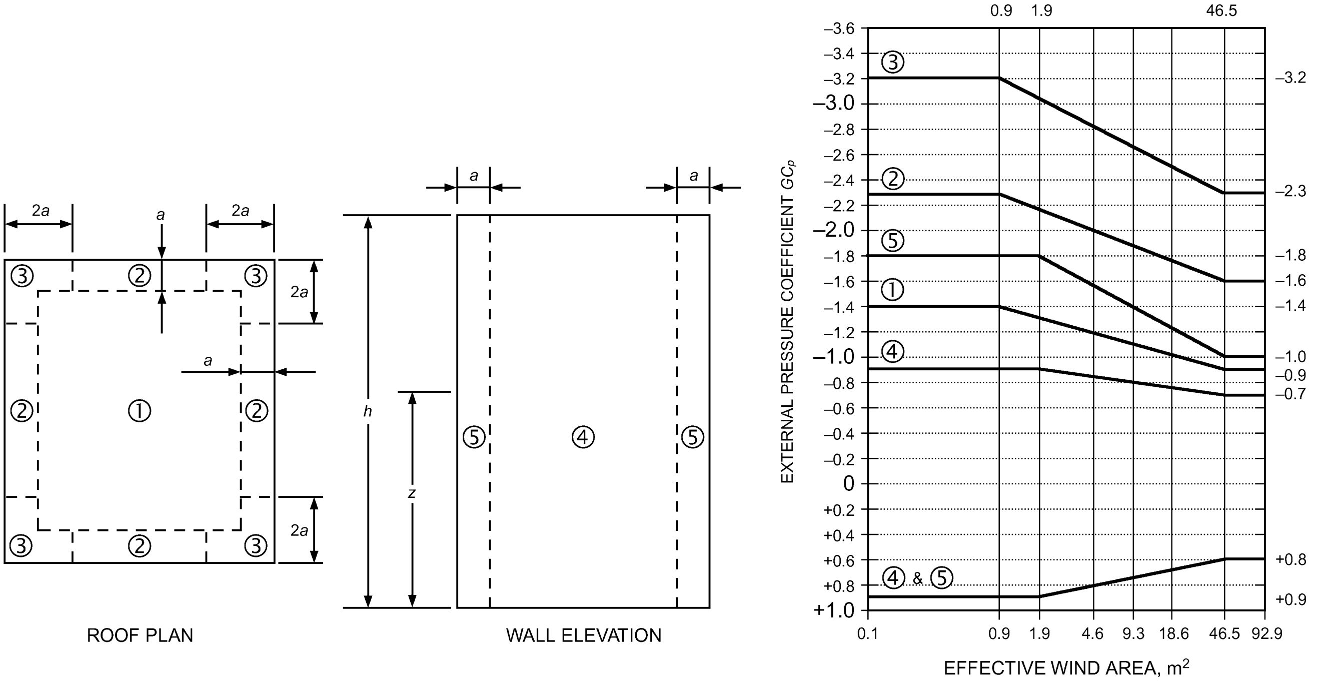
Louver 2, case 3: negative external, positive internal

$$P = 1489 \times [(-1.0) - 0.18] = -1757 \text{ Pa}$$

Louver 2, case 4: negative external, negative internal

$$P = 1489 \times [(-1.0) - (-0.18)] = -1221 \text{ Pa}$$

The controlling values for P for louver 2 are 1608 Pa, –1757 Pa and should be used to specify equipment performance levels.



NOTES:
1. Vertical scale denotes GC_p to be used with appropriate q_z or q_h.
2. Horizontal scale denotes effective wind area A, in square metres.
3. Plus and minus signs signify pressures acting toward and away from the surfaces, respectively.
4. Use q_z with positive values of GC_p and q_h with negative values of GC_p.
5. Each component shall be designed for maximum positive and negative pressures.
6. Coefficients are for roofs with angle $\theta \le 10°$. For other roof angles and geometry, use GC_p values from Figure 6-11 and attendant q_h based on exposure defined in 6.5.6.
7. If a parapet equal to or higher than 0.9 m is provided around the perimeter of the roof with $\theta \le 10°$, Zone 3 shall be treated as Zone 2.
8. Notation:
   - a: 10% of least horizontal dimension, but not less than 0.9 m.
   - h: Mean roof height, in metres, except that eave height shall be used for $\theta \le 10°$.
   - z: Height above ground, in metres.
   - θ: Angle of plane of roof from horizontal, in degrees.

Fig. 14 External Pressure Coefficient GC_p for Walls for h > 18.3 m
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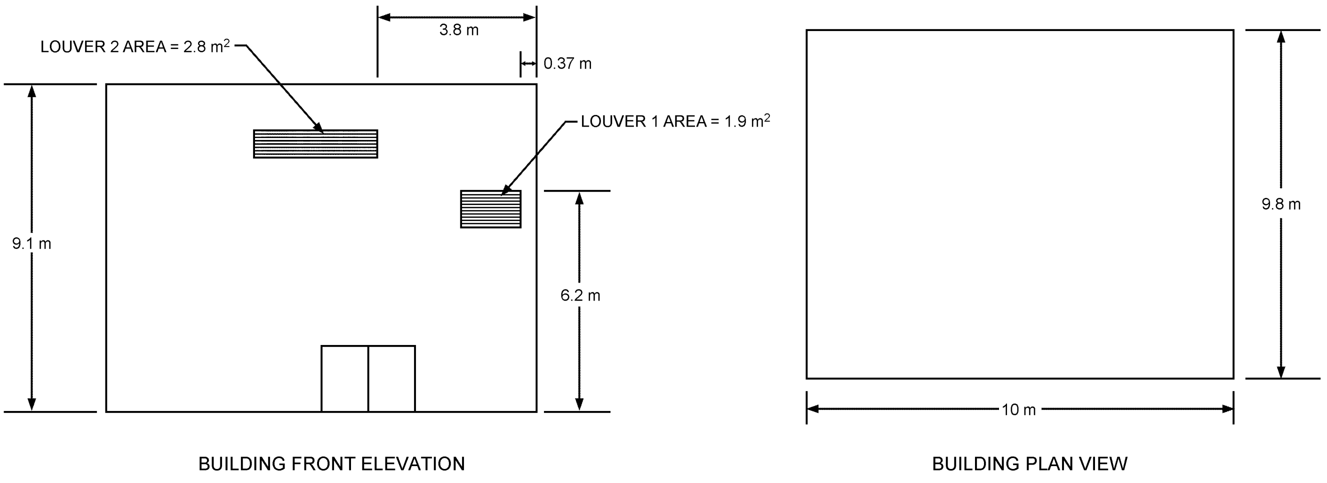

Fig. 15 Office Building, Example 10

Table 11 Classification of Buildings and Other Structures for Wind Loads

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

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2.4 CERTIFICATION OF HVAC&R COMPONENTS FOR WIND

Some jurisdictions require certifications of performance of HVAC&R components for wind resistance. These certifications focus on (1) the equipment's ability to remain intact and/or (2) the equipment restraints and anchors to keep the item in place during a wind event.

In the United States, the State of Florida and the Building Code Compliance Office of Miami-Dade County have certification requirements that affect HVAC&R system designers. The HVAC products may have special requirements for wind performance and may need approval of the State of Florida. In addition to wind performance, the Florida Building Code (ICC 2007) requires impact resistance and wind-pressure resistance for items that protect openings in buildings in windborne debris regions. The windborne debris regions can be viewed in Figure 16. HVAC products provided for projects located in these regions may be required to have testing and product certification from the State of Florida before installation. Other states, such as Texas, also have requirements for wind-pressure and impact testing. To ensure that the HVAC&R equipment

Table 12 Velocity Pressure Exposure Coefficient K_z

Height above ground level z, m	Exposure A	B	C	D
0 to 5	0.32	0.57	0.86	1.04
6	0.36	0.62	0.89	1.08
8	0.39	0.66	0.95	1.13
10	0.42	0.72	1.00	1.17
12	0.47	0.76	1.04	1.22
15	0.52	0.81	1.09	1.27
20	0.59	0.88	1.15	1.33
25	0.62	0.94	1.22	1.38
30	0.68	0.98	1.26	1.43
35	0.70	1.01	1.28	1.45
40	0.76	1.07	1.34	1.50
50	0.83	1.14	1.40	1.56
60	0.89	1.19	1.46	1.61
70	0.94	1.24	1.49	1.64
80	1.00	1.30	1.55	1.69
90	1.05	1.35	1.59	1.73
100	1.09	1.39	1.62	1.76
110	1.14	1.43	1.65	1.79
120	1.18	1.46	1.68	1.82
130	1.21	1.49	1.71	1.84
140	1.25	1.53	1.74	1.87
150	1.29	1.56	1.77	1.89

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Notes:

1. Linear interpolation for intermediate values of height z is acceptable.
2. For values of height z greater than 152.4 m, K_z must be calculated using the following equations:

$$K_z = 2.01(z/z_g)^{2/\alpha} \quad \text{For} \quad 4.5 \text{ m} \le z \le z_g$$

or

$$K_z = 2.01(15/z_g)^{2/\alpha} \quad \text{For} \quad z < 4.5 \text{ m}$$

3. Exposure categories are defined in Table 7.
4. Values for alpha (α) and z_g are found in Table 9.

Table 13 Directionality Factor K_d

Structure Type	Directionality Factor K_d*
Buildings	
Main wind-force-resisting system	0.85
Components and cladding	0.85
Arched roofs	0.85
Chimneys, tanks, and similar structures	
Square	0.90
Hexagonal	0.95
Round	0.95
Solid signs	0.85
Open signs and lattice framework	0.85
Trussed towers	
Triangular, square, rectangular	0.85
All other cross sections	0.95

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*Directionality factor K_d has been calibrated with combinations of load specified in Section 2. This factor shall only be applied when used in conjunction with load combinations specified in 2.3 and 2.4

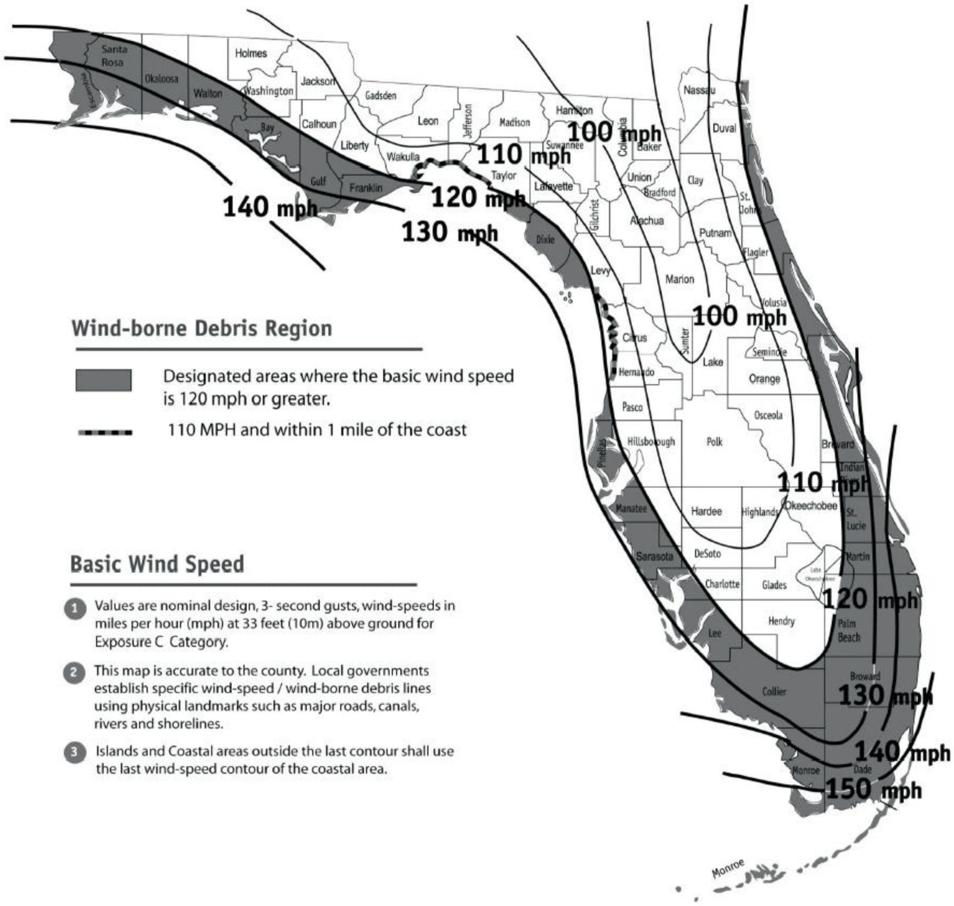

Fig. 16 State of Florida Windborne Debris Regions mph × 1.609 = km/h

ICC (2007)

Table 14 Internal Pressure Coefficient GC_{pi}

Enclosure Classification	GC_{pi}
Open buildings	0.00
Partially enclosed buildings	+0.55 –0.55
Enclosed buildings	+0.18 –0.18

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Notes:

1. Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively.
2. Values of GC_{pi} shall be used with q_x or q_h as specified in 6.5.12.
3. Two cases shall be considered to determine the critical load requirements for the appropriate condition:
   (i) a positive value of GC_{pi} applied to all internal surfaces
   (ii) a negative value of GC_{pi} applied to all internal surfaces

supplied is compliant, designers should contact the local building code official in their project location.

Some of the testing protocols for the State of Florida are

- TAS 201-94: Impact Testing Procedures
- TAS 202-94: Criteria for Testing Impact and Non-Impact Resistant Building Envelope Components Using Uniform Static Air Pressure
- TAS 203-94: Criteria for Testing Products Subject to Cyclic Wind Pressure Loading

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Related Commercial Resources