CHAPTER 1

Introduction: Understanding Loads

1.1 LOADS

In statics, loads are forces acting on structural components and are represented as uniform or varying forces or points. In practice, they represent the weights of the building materials used to construct the building (Fig. 1.1), the weights of the people and equipment which will occupy the building, and the forces of nature that the building will be exposed to during its life.

Material weights are gravity loads which act down (surprise!). **People and equipment** are primarily gravity loads, but in some instances they may cause forces which act in some other direction. An example is a piece of horizontally moving equipment which suddenly comes to a stop, such as a gantry crane, causing a horizontal force to be induced in the structure. Highway bridges are constantly subjected to this loading condition.

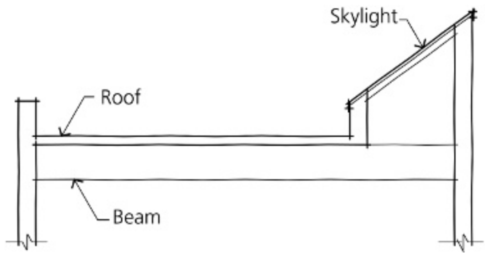
Wind forces are primarily horizontal but can induce vertical forces when blowing over surfaces. Note that wind passing over an airplane wing causes the upward lift that keeps the plane in the air. Similar conditions can be induced in the roof structure of a building.

Earthquakes, by contrast, are wave-like forces which have both horizontal and vertical components; however, the horizontal force component is typically the more destructive of the two, since most structures are designed to be primarily vertical load-carrying systems. Both wind and earthquake loads are discussed in more detail later in this chapter.

The effect of these forces is to induce states of stress and deformation or deflection in the structure. Deflections are often the governing factors in the design of a structural system. Obviously, a structure fails when it collapses; however, excessive deflection which damages finishes or other building components without causing collapse is also defined as a structural failure.

Building codes categorize these loads into two classifications: dead loads and live loads. **Dead loads** are the permanent loads generated by the constructional system. **Live loads** are the nonpermanent loads applied to the structure after it is completed. Some loads may be in either category, depending upon their **time of application.** It is essential to understand the construction sequence of the building, and to design for deflection caused by live loads introduced after the construction is complete. For example, a typically permanent (dead) load such as an HVAC (heating, ventilating, air conditioning) unit should be considered a live load if installed after ceiling finishes are in place, since it would cause deflection of the ceiling/floor components similar to that created by snow on a roof or human occupancy of a level above. This may occur even if a building component is assumed to be in place prior to finishes. A manufacturing delay, a labor dispute, a delivery problem, or even a design change may be responsible for an out-of-sequence installation which could have serious deflection implications.

An objective of the building codes is to limit the deflection of structural members to the extent that they do not damage the connected nonstructural components or affect the functionality of the building. In the 2003 International Building Code (IBC), as



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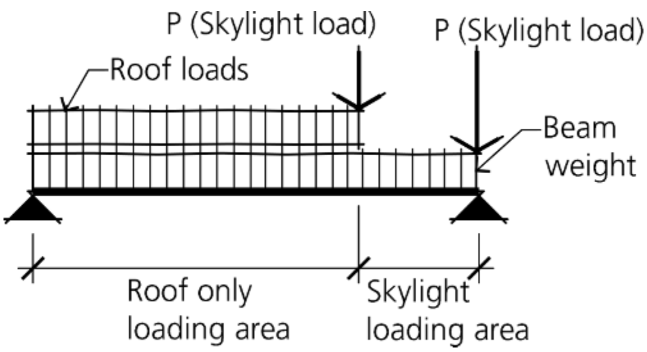


b

Figs. 1.1a, 1.1b Structural loading diagram of an architectural condition

in previous codes, limitations are imposed on deflections due to both dead and live loads (Table 1.1).

This shouldn't suggest that dead loads don't cause deflection. The dead load deflection of the structure isn't considered in some cases since it is compensated for during the construction process. For example, the ceiling finish, which is (obviously) installed after the horizontal framing is enclosed, is installed "level." Any dead load deflection which exists in the framing will be hidden by adjusting the finish. The possible exception involves roof construction; consequently, care must be taken to ensure that "flat" roof systems have no water retention areas—ponding—mentioned earlier.

The load values to be used depend on the use or occupancy of the structure. Typically, loads are floor loads, roof loads, and wind loads acting on walls and roofs, and are given in lb/ft^2 (or psf) [*kN/m^2*]. For example, floor loading for offices is 50 psf [*2.39 kN/m^2*]; for school classrooms, it is 40 psf [*1.92 kN/m^2*]. In both cases, the buildings will have corridors or circulation spaces on each level which will have a live loading of 80 to 100 psf [*3.82 to 4.78 kN/m^2*]. As this suggests, structures are subjected to a variety of live loading conditions, and the design must work for the worst-case scenario: those loading conditions which cause the worst effect on the criteria being investigated. Why are we telling you this? No matter how well you understand the processes used in designing individual members, if you begin with a misunderstanding of what loads must be resisted, how they will be applied, when they will be applied (in what

Table 1.1 Maximum Allowable Deflection for Structural Members[a]

Construction	Live Load	Snow or Wind	Live Load + K[b] Dead Load
Roof members supporting:[c]			
Rigid ceiling	L/360	L/360	L/240
Flexible ceiling	L/240	L/240	L/180
No ceiling	L/180	L/180	L/120
Floor members	L/360		L/240
Exterior/interior walls:			
Rigid finishes	—	L/240	—
Flexible finishes	—	L/120	—
Farm buildings	—	—	L/180
Greenhouses	—	—	L/120

[a] For cantilevered members, L is taken as twice the actual length of the cantilever.

[b] The value for K varies for each material discussed in this book. In Part One of this book, "Steel," the value for K is 0. This means that the maximum allowable deflection is only a function of LL and is limited to L/360 for most conditions that architects encounter. In Part Two, "Wood," the combined condition uses K = 0.5.

[c] Note that a "flat" roof member, one with a slope of $^1/_4$ in. per foot or less, should be designed for L/360, regardless of the ceiling finish, to eliminate the possibility of water ponding in the deflected areas.

combinations), and how the total system transfers these loads, your calculations will be no more than mathematical exercises.

1.1.1 How Are Loads Determined?

Gravity Loads

Dead loads are calculated by looking at the construction system of the building and calculating the actual weights of the materials. This may be difficult since preliminary calculations must be made during the design of the project, when actual materials and systems have yet to be completely defined. Knowledge of a wide variety of alternative material systems is a great asset.

Probably the best technique for determining the dead load of a building system is to sketch a typical construction section of your projections (a roof or floor "sandwich") and determine the weights of the components from manufacturers' catalogs, from tables of standard weights, or from the *AISC Manual of Steel Construction,* Table 17.13.

Of course, you will have to add any nonuniform components to this system sketch in the final calculations, such as walls, movable point loads, and so on. Note that the IBC live load table (Table 1.2) has occupancy loading criteria which include both uniform loads and movable concentrated loads; also, in the case of offices, a partition load is mandatory.

Live loads are code specified, are a result of testing, and are somewhat subjective; consequently, they are usually conservative. For example, parking garages must support a code live load of 40 psf [*1.92 kN/m²*]. If a car weighs 2,500 lb and covers an area of 14 ft $\times$ 6 ft $=$ 84 ft^2, the corresponding distributed load on the floor is 30 psf [*1.44 kN/m²*]. If people and luggage are included in the car, the load increases. The code values are conservative since there is a wide range of exceptional conditions which could occur.

Snow loads, which were previously simply a psf value based on location, are now calculated based on a number of contributing factors: C_e, exposure factor, acknowledges the influence on the surrounding terrain of snow accumulation, C_t, the thermal factor, acknowledges the melting effect during accumulation due to heat loss from the supporting structure; I_s, the importance factor, acknowledges how critical the structure is to life safety, not simply of its occupants but of the community in general; and a rain on snow surcharge consideration which acknowledges the "snow cone" effect of water trapped in snow, thereby increasing its weight. The rain on snow load of an additional 5 psf [*240 N/m²*] applies to essentially flat roof systems—1/2 in./ft [*4%*] slope maximum—in areas that have a basic snow load of 20 psf [*960 N/m²*] or less (Table 1.3).

The basic snow load formula is $P_s = 0.7C_eC_tI_sp_g$, where P_s is the design snow load, 0.7 is a basic exposure factor for flat roofs, and p_g is the geographically specified ground snow load. This load is indicated in Fig. 1.2.

The second factor of the basic snow load formula, C_t, acknowledges that the amount of heat that the structure loses will add to or subtract from the base quantity of snow retained.

Roof live loads account not only for snow, wind, or people having access for maintenance, but also for water which might accumulate if drains become clogged. The code for northern Indiana specifies 20 psf [*0.96 kN/m²*] (see Fig. 1.2), which is equivalent to 4 in. [*102 mm*] of water or as much as 40 in. [*1.2 m*] of snow. Water weighs 62.4 pcf [*9.81 kN/m³*], so every inch of water weighs 62.4 pcf/12 in./ft $=$ 5.2 psf/in. [*9.81 N/m²*] per mm of depth; 40 in. [*1.02 m*] of snow is unusual. However, it is not uncommon to find drifts of this depth, especially on roofs where building forms of different heights are adjacent to one another.

1.2 TRIBUTARY AREAS

Loads are transferred from nonstructural parts to the load-bearing structure of the building; structural members transfer the loads to each other until the forces eventually reach the foundations (Fig. 1.3). Understanding how these transfers take place is essential to assign the correct loads to all members and to design the connections between members. The location and type of load, connections, and type of structural member affect the distribution of stress (axial, bending, shear, and torque) in the cross section of the member. The assumptions made by the designer regarding the type of structural system and loads must match the reality of construction as closely as possible.

Table 1.2 Live Load Requirements: Uniform and Concentrated from the 2003 International Building Code

Occupancy or Use	Uniform Load (psf) Note 1	Concentrated (lb) Note 1
1. Apartments (see residential)	—	—
2. Access floor systems		
Office	50	2,000
Computer use	100	2,000
3. Armories and drill rooms	150	—
4. Assembly areas and theaters		—
Fixed seats (fastened to floor)	60	
Lobbies	100	
Movable seats	100	
Stages and platforms	125	
Follow spot, projection, and control rooms	50	
Catwalks	40	
5. Balconies (exterior)	100	
On one- and two-family residences only, and not exceeding 100 ft^2	60	—
6. Decks	See occupancy	—
7. Bowling alleys	75	—
8. Cornices	60	—
9. Corridors, except as otherwise indicated	100	
10. Dance halls and ballrooms	100	—
11. Dining rooms and restaurants	100	—
12. Dwellings (see residential)	—	—
13. Elevator machine room grating (on area of 4 $in.^2$)	—	30
14. Finish light floor plate construction (on area of 1 $in.^2$)	—	200
15. Fire escapes	100	—
Single-family dwellings only	40	
16. Garages		
Passenger vehicles	40	Note 2
Trucks and busses		
17. Grandstands	—	—
(see stadium and arena bleachers)		
18. Gymnasiums, main floor and balconies	100	—
19. Handrails, guard and grab bars	Note 3	
20. Hospitals		
Operating rooms, laboratories	60	1,000
Private rooms	40	1,000
Wards	40	1,000
Corridors above first floor	80	1,000
21. Hotels (see residential)		
22. Libraries		
Reading rooms	60	1,000
Stack rooms	150 Note 4	1,000
Corridors above first floor	80	1,000
23. Manufacturing		
Light	125	2,000
Heavy	250	3,000

Occupancy or Use	Uniform Load (psf) Note 1	Concentrated (lb) Note 1
24. Marquees	75	—
25. Office buildings	Note 5	
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first-floor corridors	100	2,000
Offices	50	2,000
Corridors above first floor	80	2,000
26. Penal institutions		
Cell blocks	40	—
Corridors	100	
27. Residential		
One- and two-family dwellings:		
Uninhabitable attics without storage	10	—
Uninhabitable attics with storage	20	
Habitable attics and sleeping areas	30	
All other areas except balconies/decks	40	
Hotels and multifamily dwellings		
Private:		
Rooms and corridors serving them	40	
Public:		
Rooms and corridors serving them	100	
28. Reviewing stands, grandstands, and bleachers	Note 6	
29. Roofs	Note 7	
Promenade usage	60	
Roof top gardens	100	
Assembly	100	
Landscaped	20	
Awnings	5	
30. Schools		
Classrooms	40	1,000
Corridors above first floor	80	1,000
First floor corridors	100	1,000
31. Scuttles, skylight ribs, and accessible ceilings	—	200
32. Sidewalks, vehicular driveways, and yards, subject to trucking	250 Note 8	8,000 Note 9
33. Skating rinks	100	—
34. Stadiums and arenas		—
Bleachers	100 Note 6	
Fixed seats (fastened to floor)	60 Note 6	
35. Stairs and exits (egress)	100	Note 10
One- and two-family dwellings	40	
All other	100	
36. Storage warehouses		
(shall be designed for heavier loads if required for anticipated storage)		
Light	125	
Heavy	250	
37. Stores		
Retail—First floor	100	1,000
Retail—Upper floors	75	1,000
Wholesale, all floors	125	1,000

(*continued*)

Table 1.2 Live Load Requirements: Uniform and Concentrated from the 2003 International Building Code (*continued*)

Occupancy or Use	Uniform Load (psf) Note 1	Concentrated (lb) Note 1
38. Vehicle barriers	See IBC	See IBC
39. Walkways and elevated platforms (other than egress)	60	—
40. Yards and terraces, pedestrian	100	—

For SI: 1 in. = 25.4 mm, 1 in.^{2} = 645.16 mm^2, 1 lb/ft^2 = 0.0479 kN/m^2, 1 lb = 0.004448 kN, lb/ft^3 = 16 kg/m^3.
Paraphrasing the 2003 IBC (consult the code in any case):

1. Floors shall be designed for the specified loading, either uniform or concentrated, which will create the worst loading condition for any criterion: moment, shear, deflection, or any localized mode of failure. The specified concentrated load, as set forth above, shall be placed upon any space $2^1/_2$ ft [*762 mm*] square. **It is not necessary to combine the two loading conditions.**
2. Floors in garages or portions of buildings used for the storage of motor vehicles shall be designed for the uniformly distributed live loads indicated in Table 1.1 or the following concentrated loads:
   a. For garages restricted to vehicles accommodating not more than nine passengers, 3,000 lb acting on an area of 4.5 in. by 4.5 in.
   b. For mechanical parking structures without slab or deck which are used for storing passenger vehicles only, 2,250 lb per wheel.
3. Handrail assemblies and guards shall be designed to resist the following loads:
   a. Uniform load of 50 psf [*0.73 kN/m*] applied in any direction at the top and to transfer this load through the supports to the structure.
   b. Concentrated load—Handrail assemblies and guards shall be able to resist a single concentrated load of 200 lb [*0.8 kN*] applied in any direction at any point along the top, and have attachment devices and supporting structure to transfer this load to appropriate structural elements of the building.
   c. Handrails shall be designed for the specified loading, either uniform or concentrated, which will create the worst loading condition for any criterion; moment, shear, deflection, or any localized mode of failure.

   Exceptions:
   a. For one- and two-family dwellings, handrail assemblies and guards shall be able to resist a single concentrated load of 200 lb [*0.89 kN*], applied in any direction at any point along the top, and have attachment devices and supporting structure to transfer this load to appropriate structural elements of the building.
   b. In Group I-3, F, H, and S occupancies, for areas that are not accessible to the general public and have an occupant load no greater than 50, the minimum load shall be 20 lb/ft [*0.29 kN/m*].

   Grab bars, shower seats, and dressing room bench seat systems shall be designed to resist a single concentrated load of 250 lb [*1.11 kN*] applied in any direction at any point.
4. The loading applies to stack room floors that support nonmobile, double-faced library book stacks, subject to the following limitations:
   a. The nominal book stack unit height shall not exceed 90 in.
   b. The nominal shelf depth shall not exceed 12 in. for each face, and parallel rows of double-faced book stacks shall be separated by aisles not less than 36 in. wide.
5. In office buildings and in other buildings where partition locations are subject to change, provision for partition weight shall be made, whether or not partitions are shown on the construction documents, unless the specified live load exceeds 80 psf [*3.83 kN/m^2*]. Such partition loads shall not be less than a uniformly distributed live load of 20 psf [*0.96 kN/m^2*].
6. Design in accordance with the International Code Council Standard on Bleachers, Folding and Telescopic Seating and Grandstands.
7. Roof loadings are subject to a number of alternative conditions, depending upon the slope and area. We will use a conservative but much less confusing method of applying basic roof loads to the horizontal projection of the roof. This load will be used in perpendicular and parallel components to determine bending, shear, and compressive forces in sloped roof members. For code reduction factors, see IBC Section 1607.11.2.1.
8. Other uniform loads in accordance with an approved method which contains provisions for truck loadings shall also be considered where appropriate.
9. The concentrated wheel load shall be applied on an area of 20 in.^{2}.
10. The minimum load on stair treads (on areas of 4 in.^{2}) is 300 lb.

If members support large areas of floor or roof surfaces, the likelihood of the entire area being fully loaded with live load decreases as the area increases, and code-specified live loads may be reduced based on the amount of area any individual structural member may be called upon to support. This reduction is the acknowledgment that the larger the tributary area is, the less likely that every square foot will be loaded to its maximum. For the purposes of this book, we will not incorporate this consideration and take the

Table 1.3 Snow Exposure Factor—C_e (Uniform and Concentrated from 2003 IBC [Edited])

	Exposure of Roof		
Terrain	**Fully Exposed**	**Partially Exposed**	**Sheltered**
Exposure A[a]	N/A	1.1	1.3
Exposure B[b]	0.9	1.0	1.2
Exposure C[c]	0.9	1.0	1.1
Exposure D[d]	0.8	0.9	1.0
Above treeline in mountains	0.7	0.8	N/A
Alaska[e]	0.7	0.8	N/A

[a] This exposure category is no longer used in American Society of Civil Engineers (ASCE) 7, but is still specified by IBC and is defined by areas with large buildings over 70 ft [*21.3 m*] in height.

[b] Urban and suburban areas, wooded areas, or other terrain with numerous closely spaced obstructions having the size of single-family dwellings or larger. *Exposure B shall be assumed unless the site meets the definition of another type of exposure.*

[c] Open terrain with scattered obstructions, including surface undulations or other irregularities, having heights generally less than 30 ft [*9 m*] extending more than 1,500 feet [*457.2 m*] from the building site in any quadrant. This exposure shall also apply to any building located within Exposure B-type terrain where the building is directly adjacent to open areas of Exposure C-type terrain in any quadrant for a distance of more than 600 ft [*182.9 m*]. This category includes flat open country, grasslands, and shorelines in hurricane-prone regions.

[d] Flat, unobstructed areas exposed to wind flowing over open water (excluding shorelines in hurricane-prone regions) for a distance of at least 1 mi [*1.61 km*]. Shorelines in Exposure D include inland waterways, the Great Lakes, and coastal areas of California, Oregon, Washington, and Alaska. This exposure shall apply only to those buildings and other structures exposed to the wind coming from over the water. Exposure D extends inland from the shoreline a distance of 1,500 ft [*460 m*] or 10 times the height of the building or structure, whichever is greater.

[e] In Alaska, in areas where trees do not exist within a 2 mi [*32 km*] radius of the site.

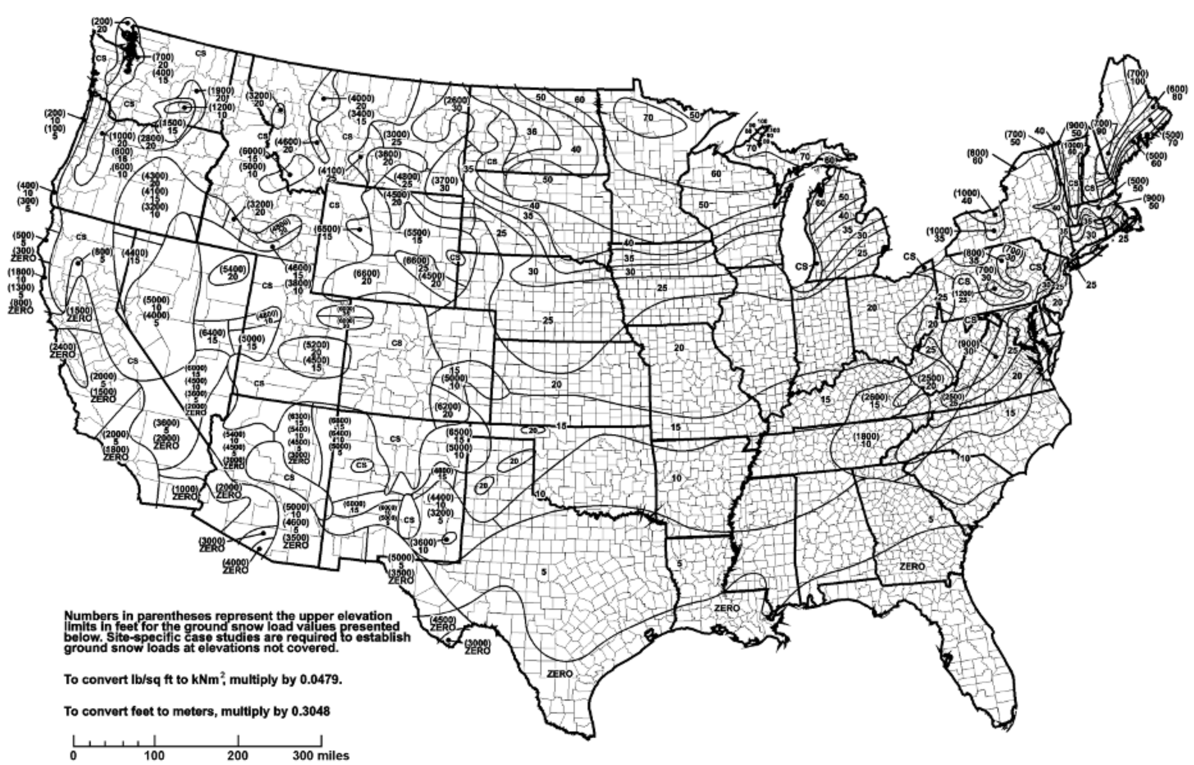


Fig. 1.2 Snow load map from the IBC

Table 1.4 Snow Thermal Factor—C_t (Uniform and Concentrated from 2003 IBS [Edited])

Thermal Condition[a]	**C_t**[b]
All structures except as indicated below	1.0
Structures kept just above freezing and others with cold, ventilated roofs in which the thermal resistance (R-value) between the ventilated space and the heated space exceeds 2.5°F(hr)/ft^2/Btu [*4.4 K(m^2)/W*]	1.1
Unheated structures	1.2
Continuously heated greenhouses with a roof having a thermal resistance (R-value) of less than 2.5°F(hr)ft^2/Btu [*4.4 K(m^2)/W*]	0.85

[a] The thermal condition shall be representative of the anticipated conditions during winters for the life of the structure.

[b] A continuously heated greenhouse shall mean a greenhouse with a constantly maintained interior temperature of 50°F [*10°C*] or more during winter months. Such a greenhouse shall also have a maintenance attendant on duty at all times or a temperature alarm system to provide warning in the event of a heating system failure.

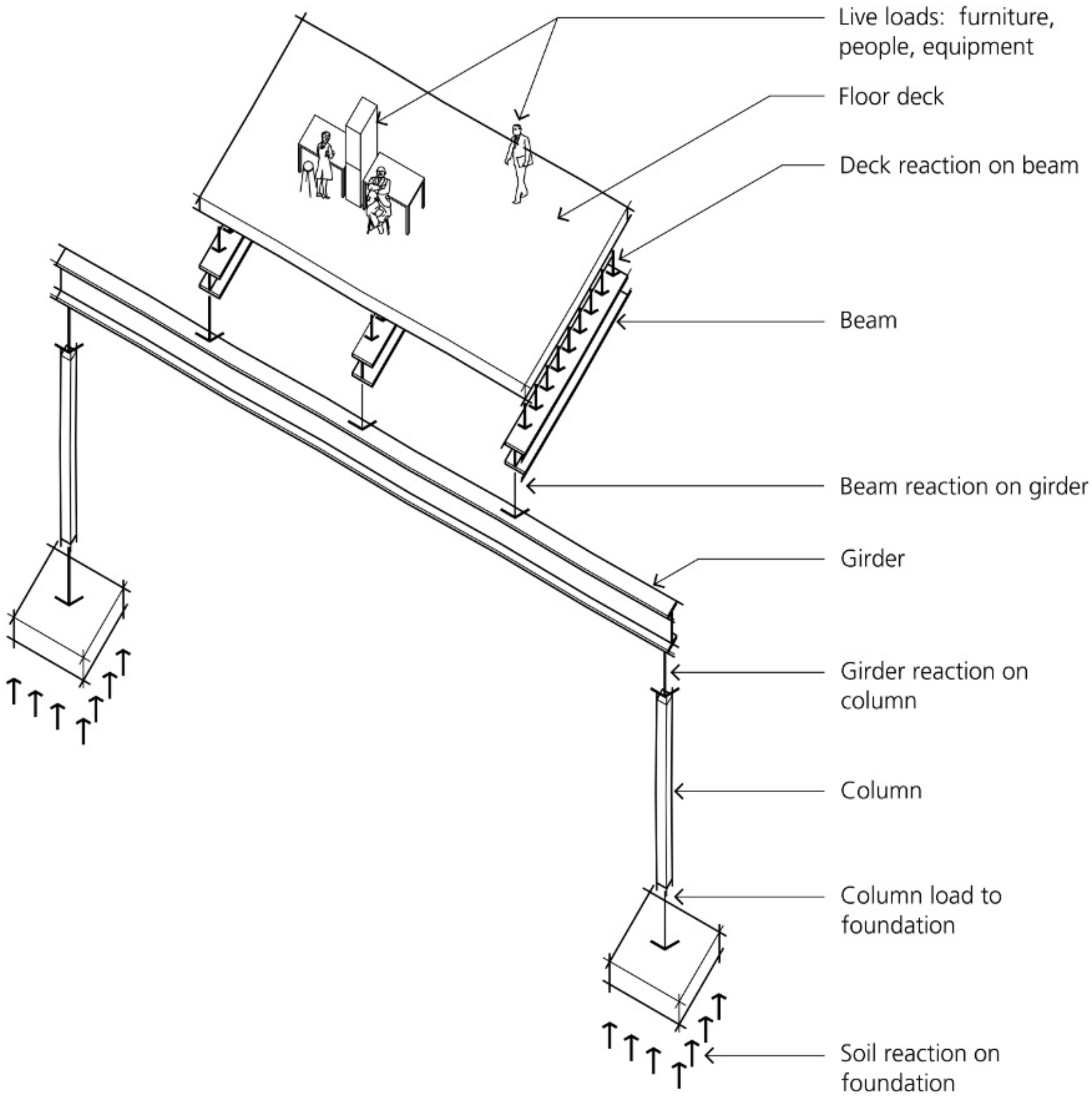


Fig. 1.3 Load paths through structures

conservative position, applying the full live load to the entire system. This conservatism will come back to haunt you when you begin to acknowledge the cost of the system, especially on large-scale buildings. If you wish further information regarding specific load reduction criteria, see Section 1607.9 of the IBC.

Dead loads do not have any area-reduction considerations.

1.3 LATERAL LOADS—WIND AND EARTHQUAKES

Lateral loads are forces acting horizontally or having a component which acts normal (perpendicular) to an inclined surface and are distinctly different from gravity loads. Lateral loads can be divided into two types: constant and variable. Lateral loads such as soil pressures on a retaining wall are relatively constant, while other loads such as wind and earthquakes have variable intensity. The code formulas use static loads in place of these dynamic actions. The magnitude of wind loads depends on the form of the structure or element, and the wind direction determines whether the load is positive (pushing) or negative (suction). Wind will exert a pressure on a roof (positive or negative) which is a function of the pitch and orientation of the roof.

Earthquakes result from a rapid release of strain energy built up between tectonic plates which make up the crust of the earth. This movement is similar to that experienced in buildings when thermal stresses cause movement at expansion joints. These joints were originally "lubricated" with brass plates to reduce friction, and now these brass plates have been replaced by Teflon-coated steel or simply Teflon pads. Teflon pads are not yet available for fault lines, so we're forced to deal with the movement and vibration caused by this enormous release of energy.

The energy release (strength of the earthquake) is measured on two types of scales: the **Modified Mercalli Intensity Scale,** ranging from 1 to 12, which evaluates the earthquake's strength based on the degree of destruction observed, and the **Richter Magnitude Scale,** which measures the magnitude of movement observed at a distance of 62 mi [*100 km*] from the point of origin (epicenter) of the quake.

The Richter Scale is a logarithmic scale ranging from 1 to 10, with each step or whole number being 30 times stronger than the previous number. This means that a magnitude 8 quake is roughly 810,000 times as strong as a magnitude 4 quake: $30 \times 30 \times 30 \times 30 = 810{,}000$. To give you a sense of these abstract numbers, a Richter magnitude 2 quake cannot be felt without instrumentation, while a magnitude 9 quake is estimated to be the largest force which can be caused by tectonic plate movement. The moon striking the earth would probably be a magnitude 10.

In reinforced concrete structures, lateral loads can be resisted by a moment-resisting frame, with columns and beams connected rigidly together. In masonry structures, the walls can provide lateral resistance when designed as shear walls (Fig. 1.4).

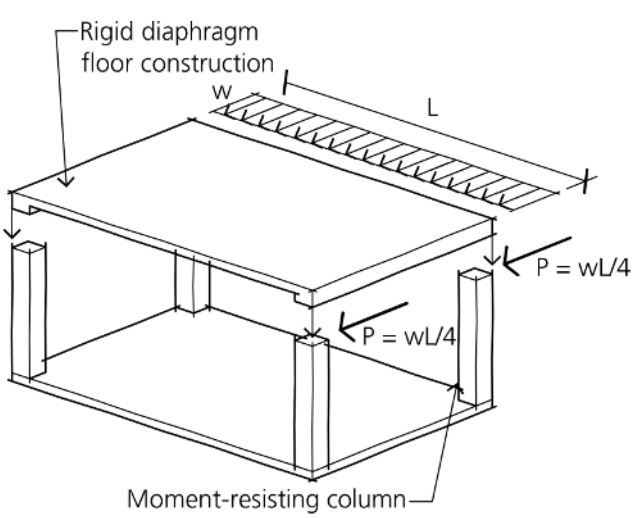


Fig. 1.4a Reinforced concrete frame

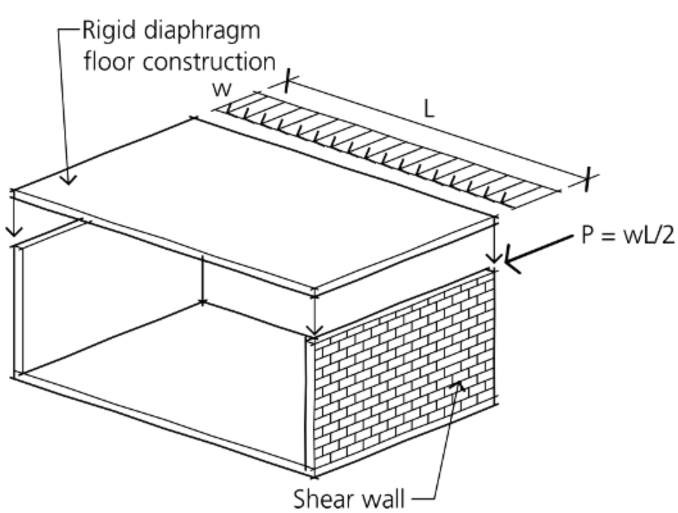


Fig. 1.4b Masonry structure using shear walls

In steel and wood frames, rigid connections are more expensive to build than in reinforced concrete; consequently, a variety of techniques ranging from rigid connections to shear walls to diagonal bracing are typically used. The structure must also be designed to resist overturning and uplift due to wind, and the same members must be designed for moment, shear, and axial load due to wind or earthquakes. These forces must be combined with the dead loads and a specified percentage of the live loads. Components such as windows and roof finishes must be designed and fastened to resist both wind pressure and wind suction. Lateral loads are assumed to act along the two principal axes of the building. If the structure is safe in these two directions, it is assumed to be adequate for loads acting in any direction. The same wind acting at an angle to a principal axis can be broken into its components acting parallel and perpendicular to the building. These components will produce only a proportion of the full force acting on the principal axis.

The problem in modern tall buildings built with a relatively light, flexible steel frame is not just strength, but also serviceability; in the case of horizontal loads, this means drift and vibrations caused by wind, earthquakes, or other dynamic loads such as machinery.

1.3.1 Structural Systems for Lateral Loads

Wind loads are transferred from the building envelope to the columns, bracing, or shear walls system anchored to the foundations. Earthquake loads similarly are assumed to act at each floor level, and the floor structure and vertical framing must be capable of transferring them to the foundations. Figure 1.5 illustrates how wind loads are resisted by vertical walls. Note that this diagram does not show all the loads on the building.

Two basic types of structural elements are used to transfer these loads through the building to the foundations (Fig. 1.6):

Horizontal Elements

- Beams in a rigid frame
- Trussed floor systems (horizontal cross-bracing)
- Floor diaphragms

Vertical Elements

- Columns in a rigid frame
- Trussed walls (vertical cross-bracing)
- Shear walls
- Rigid cores

A trussed floor is designed for horizontal loads essentially in the same way that a truss is designed for gravity loads. As for bracing, when cross-bracing is used, the diagonal members are designed to work in tension only, the most efficient use of material utilizing the smallest possible sizes. Trussed floors are common in steel construction even when concrete-filled decks are specified.

Diaphragms work as deep horizontal beams or as plates spanning between the vertical elements. A diaphragm can be classified as rigid or flexible. Common types of deck construction that can be classified as rigid diaphragms are reinforced concrete slabs, steel decks with concrete toppings, and precast concrete tees connected to resist shear. Examples of flexible diaphragms are steel decks without concrete

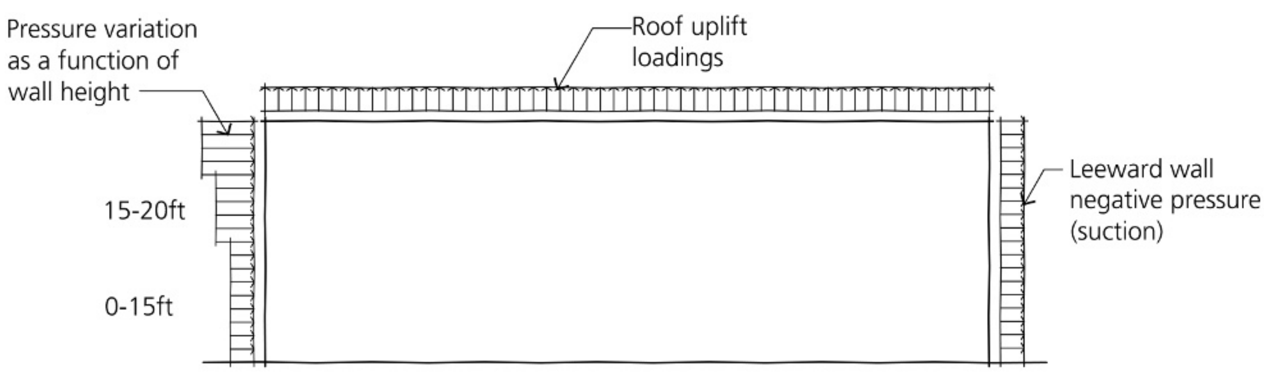


Fig. 1.5 Wind load distribution diagram

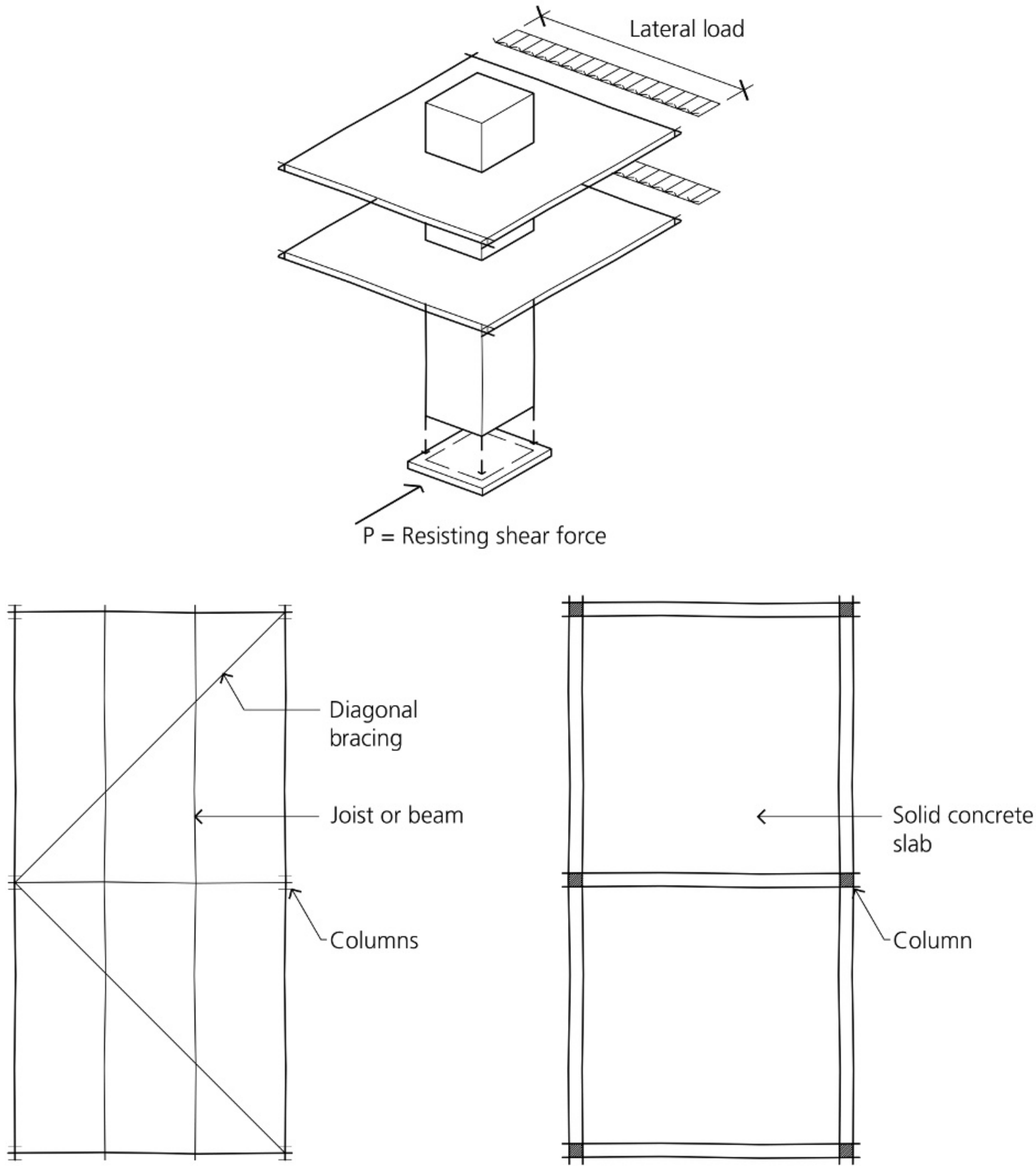


Fig. 1.6 Trussed floor and floor diaphragm systems

(common on roofs) and plywood. Rigid diaphragms can resist torsion, and some eccentricity in the design of vertical elements is possible. Flexible diaphragms, however, require vertical shear supports at both sides in each direction (Fig. 1.7).

Some steel decks can be considered rigid diaphragms, depending on the gage (thickness of the steel) and depth. Manufacturers publish data on the use of their deck as a diaphragm, particularly the shear strength and the degree of flexibility. The connection between beams and decking is critical. This is accomplished with welds on each panel placed where the deck rib rests on the beam flange. In addition, deck panels should be attached to each other by side-seam welds, or screws placed at intervals not exceeding 36 in. [*914 mm*]. In any event, the specific

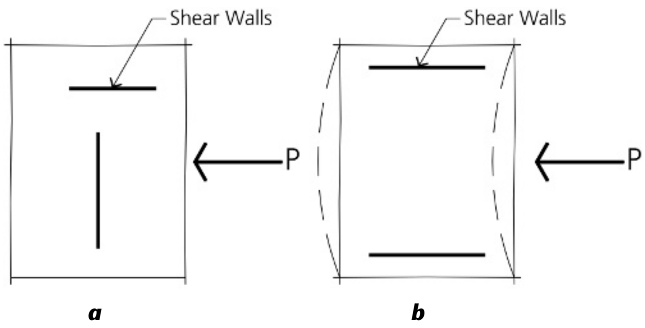


Fig. 1.7 (a) Rigid diaphragm; (b) flexible diaphragm

manufacturer must be consulted and the recommendations adhered to.

Horizontal diaphragms transfer their load to a braced vertical structural element. In steel systems this element might be a cantilevered column, a solid cantilevered wall, a trussed cantilevered structure, a rigid frame with moment connections, or even a simple diagonal bracing system. In concrete frames, moment connections are easier to create than to avoid. Consequently, they may or may not utilize additional visually observable bracing systems. In order to resist lateral loads, we have basically four techniques or combinations of techniques.

1. Short cantilever, or shear wall, working primarily in (surprise!) shear.
2. Long cantilever: rigid three-dimensional core, tall shear walls, or a pierced tube (a core with holes) acting as a beam or truss working in bending as well as shear.
3. A trussed cantilever, as the name indicates, is a vertical truss acting in bending and shear but developing primarily axial loads in tension or compression (a truss).
4. A rigid frame, a standard structural system of columns and beams with moment connections to provide lateral stability. The moment connections cause all members to be subject to combined axial load and bending. These are statically indeterminate structures and are typically analyzed on a computer.

The combination of a frame and a wall or core element is called a "dual" system. You'll see this referred to in the IBC discussions of framing systems. The types of framing systems that can be designed for lateral loads are endless, and decisions depend on architectural considerations.

The code provisions for reinforced-concrete buildings in seismic areas assume the use of rigid "moment-resisting" frames with or without shear walls, and the prescriptions mainly concern reinforcement details and maximum stress values. Some of the earthquake issues that are easily considered in preliminary design are minimum sizes of beams and columns. In seismic zones 3 and 4, beams must be at least 10 in. [*254 mm*] wide, and the width-to-depth ratio must be at least 0.3; that is, the minimum beam would be 10 in. $\times$ 33.3 in. [*254 mm* $\times$ *846 mm*]. For columns, the smallest dimension in cross section must be 12 in. [*304 mm*]. The column requirement is mainly to ensure that enough space is provided for the necessary reinforcing bars and ties.

Although we have discussed primarily steel and concrete systems, most buildings are a combination of systems. While concrete and steel systems can be used in almost any combination, the code does not allow the use of concrete or masonry systems on wood support structures, although horizontal wood structures may be used on concrete or masonry supports. The notable exception is that steel joist systems may be supported on wood columns or walls.

1.4 LATERAL LOADS: WIND

Lateral loads act horizontally and are typically dynamic. They push and pull and under the most extreme conditions can cause rhythmic loading, which amplifies their effect. Consider the rhythmic application of force which causes the amplitude of your motion to increase in a simple child's swing. Wind exerts a positive pressure or a negative pressure (suction) on the exposed surfaces of buildings. The magnitude of the forces depend on a number of factors; the most important are wind velocity, exposure, and building shape.

Wind velocity changes continuously with time, but for our purposes it is defined with an average value. The effects of dynamic gust velocity depend on the stiffness of the structure. Building codes in general quantify wind pressure using a nominal design 3-second gust wind speed in miles per hour measured at a point 33 ft [*10 m*] above the ground. Linear interpolation between wind contours on wind speed maps is permitted. Note that Figure 1.8 includes the general values for the entire United States, and as you can see, along coastal areas the contours become very dense. For this reason, the IBC provides supplementary maps at a larger scale for these regions. The 3-second gust speeds are converted to the fastest-mile wind velocities and are shown in Figure 1.8.

Wind velocity is reduced by friction from the irregularities of the terrain. Above certain heights, the air can move without being affected by ground conditions. This height is called the "gradient height," and the velocity is called *q* "wind speed" (Fig. 1.9).

1.4.1 Wind Design Criteria

There are several different methods for calculating wind forces on buildings, and as you may imagine, the most sophisticated ones are very complicated. The

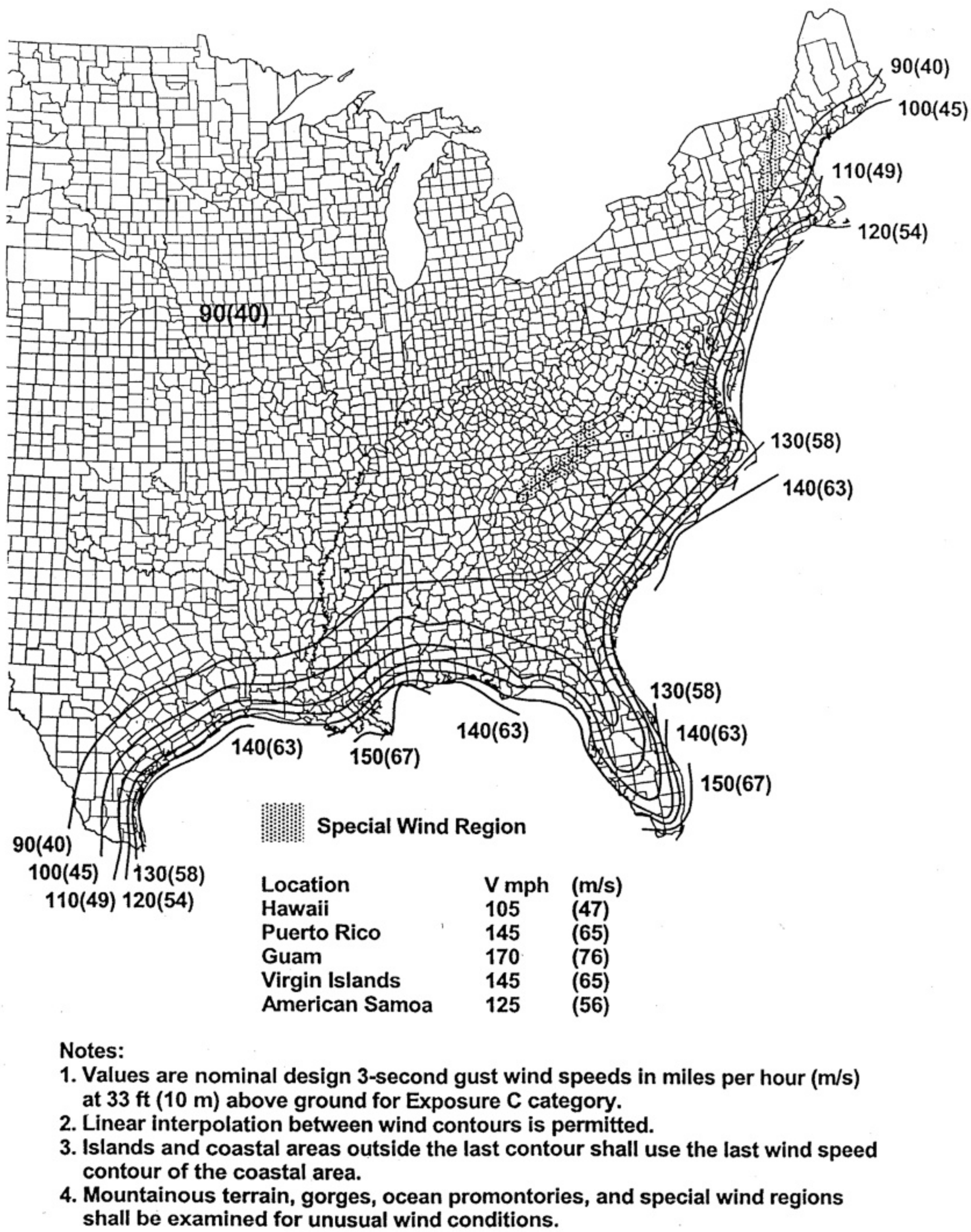


Fig. 1.8 Section of a gust wind velocity map from the IBC

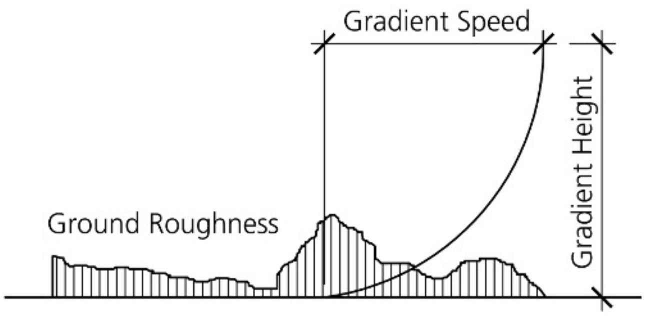


Fig. 1.9 A wind velocity gradient

latest version of the IBC has methodologies that are very sophisticated and require a qualified structural engineer. At the same time, architects have to know the IBC code requirements, and they need a methodology for daily use for fundamental calculations. Our approach is to identify the relevant issues for the IBC and to provide the Uniform Building Code (UBC) methodology, which, although not as specific, gives very similar force values and can be used by the typical architect. We have included parallel IBC calculations for the examples on our web site for those of you who have an interest.

The IBC introduces several new considerations into its evaluation of wind loads;

1. Gust factor effect. Wind does not blow at a steady rate, and wind gusts can have a significant impact in some structural situations. This is one of the factors that led to the collapse of the Tacoma Narrows ("Galloping Gertie") bridge in Tacoma, WA, in November 1940. While the causes of the collapse are still argued, it is clear that aerodynamic phenomena greatly influenced it.

2. Terrain factors. If you have ever walked outdoors or driven down a road on a very windy day, you have noticed that around a corner in a city, the wind may dramatically change. Or, as you drive out of a wooded area into a more open area, your car may swerve sideways. This movement shows that your exposure to the wind is being influenced by localized terrain features. The IBC introduces factors that take into account features of the local terrain—such as hills and escarpments—that increase or decrease the impact of the wind.

3. Building surface location. For low buildings, up to 60 ft [*18.3 m*] in height, the wind will create different forces on different areas of the building's surface. The wind will tend to stagnate at interior locations and will have more effect at the ends of walls and roofs, where the wind is being directed around the building and will experience higher pressures. This makes sense when we consider that wind is a form of fluid movement; the Venturi effect will cause higher wind velocities and consequently higher pressures at points where wind flow is constricted.

While the overall forces may be quite similar to those in the previous UBC methodologies, the impact on specific areas of the structure may be different. For a very precise investigation, the IBC procedure is advisable.

4. Localized wind conditions. The IBC has included much more specific wind speed maps for a number of locations in the United States. These are obviously the result of additional data collection over the years, as well as of the recognition of the unique circumstances associated with coastal regions.

For conditions of loading that are influenced by natural forces, not just people and materials, the IBC has created occupancy importance factors which acknowledge the relative importance or relative negative impact of potential failure due to loading conditions for earthquakes, snow, and wind. Although these are quite similar to those specified by the UBC, they reflect increased concern for the stability of essential facilities such as hospitals, emergency and life safety facilities, jails, and so on. These factors are listed in Table 1.5.

1.4.2 General Building Design—UBC Methodology

The UBC defines three degrees of exposure, which are almost identical to those of the IBC:

Exposure B has terrain with buildings, forest, or surface irregularities 20 ft [*6 m*] or more in height covering at least 20% of the area extending 1 mile [*1.6 km*] or more from the site.

Exposure C has terrain which is flat and generally open, extending $^1/_2$ mile [*0.8 km*] or more from the site in any full quadrant.

Exposure D represents the most severe exposure in areas with basic wind speeds of 80 mph [*130 km/h*] or greater and has terrain which is flat and

Table 1.5 IBC Occupancy Importance Factors for Earthquake (I_E), Snow (I_S), and Wind (I_W)

Category[a]	Nature of Occupancy	Seismic Factor I_E	Snow Factor I_S	Wind Factor I_W
I	Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: a. Agricultural facilities b. Certain temporary facilities c. Minor storage facilities	1.00	0.80	0.87[b]
II	Buildings and other structures except those listed in Categories I, III, and IV	1.00	1.00	1.00
III	Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: a. Buildings and other structures where more than 300 people congregate in one area b. Buildings and other structures with elementary school, secondary school, or day care facilities with an occupant load greater than 250 c. Buildings and other structures with an occupant load greater than 500 for colleges or adult education facilities d. Health care facilities with an occupant load of 50 or more resident patients but without surgery or emergency treatment facilities e. Jails and detention facilities f. Any other occupancy with an occupant load greater than 5,000 g. Power-generating stations, water treatment facilities for potable water, wastewater treatment facilities, and other public utility facilities not included in Category IV h. Buildings and other structures not included in Category IV containing sufficient quantities of toxic or explosive substances to be dangerous to the public if released	1.25	1.10	1.15
IV	Buildings and other structure designed as essential facilities including, but not limited to: a. Hospitals and other health care facilities having surgery or emergency treatment facilities b. Fire, rescue, and police stations and emergency vehicle garages c. Designated earthquake, hurricane, or other emergency shelters d. Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response e. Power-generating stations and other public utility facilities requires as emergency backup facilities for Category IV structures f. Structures containing highly toxic materials as defined in Section 307 of the IBC where the quantity of the material exceeds the maximum allowable quantities of Table 307.7(2) of the IBC g. Aviation control towers, air traffic control centers, and emergency aircraft hangars h. Buildings and other structures having critical national defense functions i. Water treatment facilities required to maintain water pressure for fire suppression	1.50	1.20	1.15

Note: For buildings that have more than one occupancy group the highest category factor will govern the design.

[a] Categories I and II are considered Seismic Use Group I, Category III is considered Seismic Use Group II, and Category IV is considered Seismic Use Group III.

[b] In hurricane-prone regions with wind velocities of 100 mph, I_w shall be 0.77.

unobstructed facing large bodies of water over 1 mi [*1.6 km*] or more in width relative to any quadrant of the building site. Exposure D extends inland from the shoreline $^1/_4$ mi [*0.4 km*] or 10 times the building height, whichever is greater.

The minimum basic wind speed for determining wind pressure can be found in Fig. 1.10 of the 1997 UBC, which is reproduced here. Note that this wind pressure map is significantly less refined than the IBC maps we have previously reviewed.

Design wind loads are calculated with the formula from the UBC:

$$P = C_e C_q q_s I_w$$

where

C_e = combined height, gust factor, and exposure coefficient (Table 1.6)

C_q = pressure coefficient, depending on the type of structure (Table 1.7)

q_s = wind stagnation pressure (Table 1.8)

I_w = importance factor, depending on the occupancy category (Table 1.9)

P = pressure measured in psf

Note: When you investigate the code, you will see that the UBC refers to two ways of determining loads: the **normal force method** and the **projected area method.** These methods will ultimately yield the same answers for any condition, but they can be used with some forethought to learn what you need to know as easily as possible. The normal force method will give you directly the force which acts normal, or perpendicular, to any surface, either inward or outward. This would be a desirable technique if you were designing a sloped roof joist, since you would immediately determine the force acting perpendicular to the joist which creates moment, shear, and deflection in the joist. You would need to calculate the component of force which acts

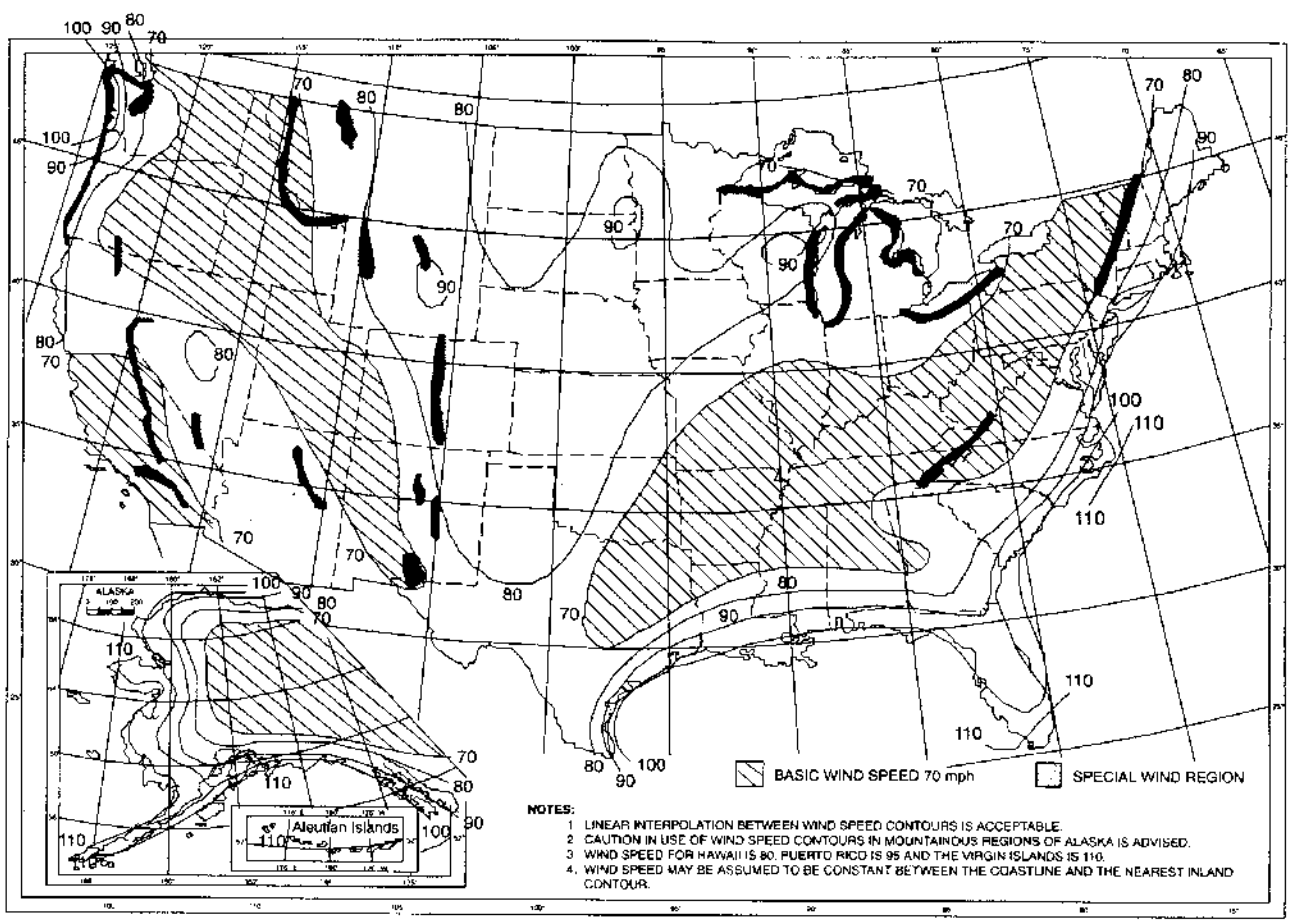


Fig. 1.10 Wind speed map for the United States

Table 1.6 C_e Combined Height, Exposure, and Gust Factor Coefficient

Height Above Average Level of Adjoining Ground (ft) [*m*]	Exposure D	Exposure C	Exposure B
0–15 ft. [*0–4.6 m*]	1.39	1.06	0.62
20 ft. [*6.1 m*]	1.45	1.13	0.67
25 ft. [*7.6 m*]	1.50	1.19	0.72
30 ft. [*9.1 m*]	1.54	1.23	0.76
40 ft. [*12.2 m*]	1.62	1.31	0.84
60 ft. [*18.3 m*]	1.73	1.43	0.95
80 ft. [*24.4 m*]	1.81	1.53	1.04
100 ft. [*30.5 m*]	1.88	1.61	1.13
120 ft. [*36.6 m*]	1.93	1.67	1.20
160 ft. [*48.8 m*]	2.02	1.79	1.31
200 ft. [*61 m*]	2.10	1.87	1.42
300 ft. [*91.5 m*]	2.23	2.05	1.63
400 ft [*122 m*]	2.34	2.19	1.80

Note: Values for intermediate heights above 15 ft [*4.6 m*] may be interpolated, i.e., for height = 22.5 ft [*6.9 m*], Exposure D, $C_e = 1.45 + (1.50 - 1.45)/2 = 1.475$. That was obviously easy, but you get the idea. As you might logically expect, the effect of height tends to negate the effect of terrain. As you get higher, the numbers have much less differential.

Table 1.7 C_q (1997 UBC Edited) Pressure Coefficient

Structure of Part Thereof	Description	C_q Factor
Primary frames and systems	**Method 1 (normal force method)**	
	Walls:	
	Windward wall	0.8 inward
	Leeward wall	0.5 outward
	Roofs:[a]	
	Wind perpendicular to ridge	
	Leeward roof or flat roof	0.7 outward
	Windward side:	
	Less than 2:12 slope	0.7 outward
	Slope 2:12 < 9:12	0.9 outward or 0.3 inward
	Slope 9:12 < 12:12	0.4 inward
	Slope > 12:12	0.7 outward
	Wind parallel to ridge/flat roofs	0.7 outward
	Method 2 (projected area method)	
	On vertical projected area	
	Structures 40 ft [*12.2 m*] or less in height	1.3 horizontal in any direction
	Structures over 40 ft [*12.2 m*] in height	1.4 horizontal in any direction
	On horizontal projected area	0.7 upward

(*continued*)

Table 1.7 C_q **(1997 UBC Edited) Pressure Coefficient (*continued*)**

Structure of Part Thereof	Description	C_q Factor
Elements and components not in areas of discontinuity[b]	**Wall elements** All structures Enclosed and unenclosed structures Partially enclosed structures Parapet walls **Roof elements**[c] Enclosed and unenclosed structures Slope < 7:12 Slope > 7:12 to 12:12 **Partially enclosed structures**[c] Slope < 2:12 Slope 2:12 to 7;12 Slope > 7:12 to 12:12	 1.2 inward 1.2 outward 1.6 outward 1.3 inward or outward 1.3 outward 1.3 outward or inward 1.7 outward 1.6 outward or 0.8 inward 1.7 outward or inward
Elements and components in areas of discontinuity[b,d,e]	Wall corners[f] Roof eaves, rakes, or ridges without overhangs[f] Slope < 2:12 Slope 2:12 to 7:12 Slope > 7:12 to 12:12 For slopes less than 2:12, overhangs at roof eaves, rakes or ridges, and canopies	1.4 outward or 1.2 inward 2.3 upward 2.6 outward 1.6 outward 0.5 added to values above
Chimneys, tanks, and solid towers	Square or rectangular Hexagonal or octagonal Round or elliptical	1.4 in any direction 1.1 in any direction 0.8 in any direction
Open-frame towers[g,h]	Square or rectangular Diagonal Normal Triangular	 4.0 3.6 3.2
Signs, flagpoles, light poles, minor structures[h]		1.4 in any direction

Note: Using SI units, the slopes are more commonly expressed at percentages, e.g., 7:12 = 0.583 or 58.3%.

[a] For one-story structures or the top story of multistory, partially enclosed structures, an additional value of 0.5 shall be added to the outward C_q. The most critical combination shall be used for design. For definition of partially enclosed structures see Section 1616 of the 1997 UBC.

[b] C_q values listed are for 10 ft^2 [*0.93 m*^{2}] areas. For tributary areas of 100 ft^2 [*9.29 m*^{2}], the value of 0.3 may be subtracted from C_q, except for areas at discontinuities with slopes less than 7:12, where the value of 0.8 may be subtracted from C_q. Interpolation may be used for tributary areas between 10 and 100 ft^2 [*0.93 and 9.29 m*^{2}]. For tributary areas greater than 1,000 ft^2 [*92.9 m*^{2}], use primary frame values.

[c] For slopes greater than 12:12, use wall element values.

[d] Local pressures shall apply over a distance from the discontinuity of 10 ft [*3 m*] or 0.1 times the least width of the structure, whichever is smaller.

[e] Discontinuities at wall corners or roof ridges are defined as discontinuous breaks in the surface where the included interior angle measures 170° or less.

[f] The load is to be applied on either side of the discontinuity but not simultaneously on both sides.

[g] Wind pressures shall be applied to the total normal projected area of all elements on one face. The forces shall be assumed to act parallel to the wind direction.

[h] Factors for cylindrical elements are two-thirds of those for flat or angular elements.

Table 1.8 q_s **Wind Stagnation Pressure at a Standard Height of 33 ft [*10 m*]**

Basic wind speed (mph)[a] [× 1.61 for kmh]	70	80	90	100	110	120	130
Pressure q_s (psf) [× 0.0479 for kN/m^2 or Pa]	12.6	16.4	20.8	25.6	31.0	36.9	43.3

[a] Wind speed from Fig. 1.10.

Table 1.9 I, I_p Seismic Importance Factor; I_W Wind Importance Factor

Category	Occupancy or Functions of Structure	Seismic I	Semis I_p[a]	Wind I_w
I_p Essential facilities[b]	Group 1, Division I occupancies having surgery and emergency-treatment areas Fire and police stations Garages and shelters for emergency vehicles and emergency aircraft Structures and shelters in emergency-preparedness centers Aviation control towers Structures and equipment in government communication centers and other facilities required for emergency response Standby power-generating equipment for Category 1 facilities Tanks or other structures containing housing or supporting water or other fire-suppression material or equipment required for the protection of Category 1, 2, or 3 structures	1.25	1.50	1.15
Hazardous facilities	Group H, Divisions 1, 2, 6, and 7 occupancies and structures therein housing or supporting toxic or explosive chemicals or substances Nonbuilding structures housing, supporting, or containing quantities of toxic or explosive substances that, if contained within a building, would cause that building to be classified as a Group H, Division 1, 2, or 7 occupancy	1.25	1.50	1.15
Special occupancy structures[c]	Group A, Division 1, 2, or 2.1 occupancies Buildings housing Group E, Divisions, 1 and 3 occupancies with a capacity greater than 300 students. Buildings housing Group B occupancies used for college or adult education with a capacity greater than 500 students Group I, Division 1 and 2 occupancies with 50 or more resident incapacitated patients, but not included in Category 1 Group I, Division 3 occupancies All structures with an occupancy greater than 5,000 persons Structures and equipment in power-generation stations, and other public utility facilities not included in preceding Category 1 or 2 and required for continued operation	1.00	1.00	1.00
Standard occupancy structures[c]	All structures housing occupancies or having functions not listed in Category 1, 2, or 3 and Group U occupancy towers	1.00	1.00	1.00
Miscellaneous structures	Group U occupancies except for towers	1.00	1.00	1.00

Note: These importance factors vary from those of the IBC presented in Table 1.5.

[a] The limitation of I_p for panel connections in Section 1633.2.4 of the UBC shall be 1.0 for the entire connector.

[b] Structural observation requirements are given in Section 1702 of the UBC.

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

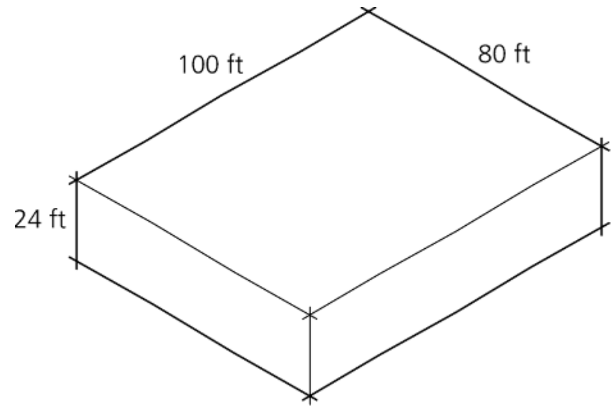
While it would appear that categories 1 and 2 and categories 3, 4, and 5 could be combined into two simple groups, a distinction is likely to accommodate future changes in the code.

EXAMPLE 1.1

A school in a flat urban area in Indianapolis, Indiana, with a footprint of 80 ft $\times$ 100 ft, with a wall height of 24 ft, flat roof, Exposure B.

Using the UBC approach, determine the total horizontal and vertical forces acting on this building using the normal force method, which would be most appropriate for designing individual members or components of the structure.

$P = C_e C_q q_s I_w$



(a) Positive Pressure: Inward

Exposure Coefficients:

$C_e = 0.62$ 0–15 ft (Table 1.6)
$C_e = 0.67$ 15–20 ft
$C_e = 0.72$ 20–25 ft

Pressure Coefficient:

$C_q = 0.8$

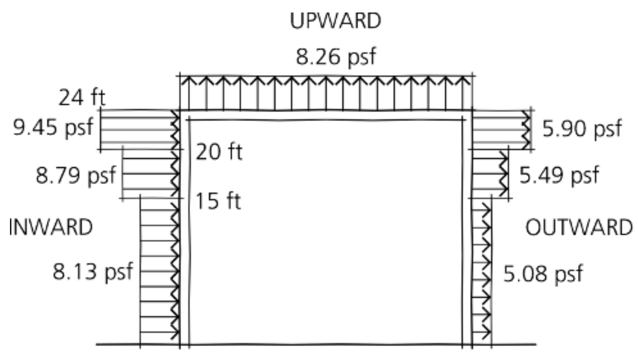
Basic wind speed from Table 16.1.10 = 80 mph; therefore, $q_s = 16.4$ psf

$I_w = 1.0$ Special occupancy structure (Table 1.9)

Pressure:

0–15 ft $P_1 = 0.62 \times 0.8 \times 16.4 \text{ psf} \times 1.0 = 8.13 \text{ psf}$
15–20 ft $P_2 = 0.67 \times 0.8 \times 16.4 \text{ psf} \times 1.0 = 8.79 \text{ psf}$
20–25 ft $P_3 = 0.72 \times 0.8 \times 16.4 \text{ psf} \times 1.0 = 9.45 \text{ psf}$

(b) Negative Pressure: Outward



Exposure Coefficients:

$C_e = 0.62$ 0–15 ft (Table 1.6)
$C_e = 0.67$ 15–20 ft
$C_e = 0.72$ 20–25 ft

Pressure Coefficient:

$C_q = 0.5$

Pressure:

0–15 ft $P_1 = 0.62 \times 0.5 \times 16.4 \text{ psf} \times 1.0 = 5.08 \text{ psf}$
15–20 ft $P_2 = 0.67 \times 0.5 \times 16.4 \text{ psf} \times 1.0 = 5.49 \text{ psf}$
20–25 ft $P_3 = 0.72 \times 0.5 \times 16.4 \text{ psf} \times 1.0 = 5.90 \text{ psf}$

(c) Roof Uplift: Outward

$C_e = 0.72$ (based on mean roof height = 24 ft, which puts us in the 20–25 ft range)
$C_q = 0.7$
$P_4 = 0.72 \times 0.7 \times 16.4 \text{ psf} \times 1.0 = 8.26 \text{ psf}$

These pressures must be applied simultaneously perpendicular to the walls and to the roof surfaces. The vertical loads can be considered to resist overturning and suction.

Note that these pressures are lower than the ones obtained using the IBC procedures. This is because the IBC uses a 90 mph wind classification for this region, and the UBC uses 80 mph. Additionally, the IBC uses a higher value at the end zones to account for the compression of the wind as it "rounds the corners" and this factor is not considered in the UBC versions.

EXAMPLE 1.2

We'll now solve a different problem with an 80 ft tall, slender office building which has a total weight (worst condition with DL only) of 1,423 kips and a footprint of 40 ft × 100 ft, again using the UBC approach but the projected area method. This method is best used when evaluating the overall "overturning" forces on the building, which will likely become an issue in a taller structure with a relatively narrow base.

The building weighs 1,423 kips and, with a 40 ft base, assuming that it will "topple" about the narrow dimension (a pretty clear option), it has an overturning resisting moment (weight × moment arm) of 1,423 k (40 ft/2) = 28,460 ftk. This is the force which would hold it in place under high wind loads.

The overturning forces can be found by calculating the overturning wind forces acting on the structure.

(a) Horizontal Pressure

EXPOSURE COEFFICIENTS:

$C_e = 0.62$ 0–15 ft (Table 1.6)
$C_e = 0.67$ 15–20 ft
$C_e = 0.72$ 20–25 ft
$C_e = 0.76$ 25–30 ft
$C_e = 0.84$ 30–40 ft
$C_e = 0.95$ 40–60 ft
$C_e = 1.04$ 60–80 ft

PRESSURE COEFFICIENT:

$C_q = 1.4$

(b) Total Gross Pressure (Inward + Outward)

Basic wind speed from Fig. 1.10 = 80 mph; therefore, q = 16.4 psf

$I_w = 1.0$ primary frames and systems (Table 1.9)

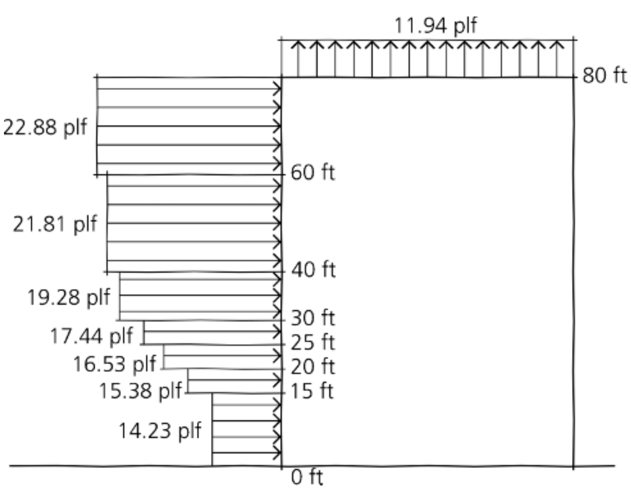
PRESSURE:

0–15 ft	$P_1 = 0.62 \times 1.4 \times 16.4$ psf $\times 1.0 = 14.23$ psf
15–20 ft	$P_2 = 0.67 \times 1.4 \times 16.4$ psf $\times 1.0 = 15.38$ psf
20–25 ft	$P_3 = 0.72 \times 1.4 \times 16.4$ psf $\times 1.0 = 16.53$ psf
25–30 ft	$P_4 = 0.76 \times 1.4 \times 16.4$ psf $\times 1.0 = 17.44$ psf
30–40 ft	$P_5 = 0.84 \times 1.4 \times 16.4$ psf $\times 1.0 = 19.28$ psf
40–60 ft	$P_6 = 0.95 \times 1.4 \times 16.4$ psf $\times 1.0 = 21.81$ psf
60–80 ft	$P_7 = 1.04 \times 1.4 \times 16.4$ psf $\times 1.0 = 23.88$ psf

If you take the values for positive and negative pressure acting on opposite sides of the building from the problem above, you will note that on the positive side, the C_e factor is 0.8 and on the negative side it is 0.5, which total 1.3; this is the C_e factor for buildings under 40 ft using the projected area method. For taller buildings, there is a difference since the C_e factor becomes 1.4, as we saw in this problem.

This suggests that if you want only the total horizontal forces, the projected area method gives you the same answers with 50% of the work. Take your choice.

(c) Roof Uplift



EXPOSURE COEFFICIENT:

$C_e = 1.04$ (using a roof height of 80 ft)

PRESSURE COEFFICIENT:

$C_q = 0.7$

PRESSURE:

$P_{roof} = 1.04 \times 0.7 \times 16.4$ psf $\times 1.0 =$ 11.94 psf

To calculate the overturning moment, we determine the total force acting at the centroid of each level and multiply it by its moment arm relative to the base.

TOTAL WIND FORCE AT EACH ZONE:

0–15 ft	$F_1 = 14.23$ psf $\times$ (15 ft high $\times$ 100 ft long) = 21,345 lb = 21.35 kips
15–20 ft	$F_2 = 15.38$ psf $\times$ (5 ft high $\times$ 100 ft long) = 7,690 lb = 7.69 kips
20–25 ft	$F_3 = 16.53$ psf $\times$ (5 ft high $\times$ 100 ft long) = 8,265 lb = 8.27 kips
25–30 ft	$F_4 = 17.44$ psf $\times$ (5 ft high $\times$ 100 ft long) = 8,720 lb = 8.72 kips
30–40 ft	$F_5 = 19.28$ psf $\times$ (10 ft high $\times$ 100 ft long) = 19,280 lb = 19.28 kips

40–60 ft $F_6 = 21.81$ psf $\times$ (20 ft high $\times$ 100 ft long) $= 43{,}620$ lb $= 43.62$ kips
60–80 ft $F_7 = 23.88$ psf $\times$ (20 ft high $\times$ 100 ft long) $= 47{,}760$ lb $= 47.62$ kips
Roof uplift $= 11.94$ psf $\times$ (40 ft wide $\times$ 100 ft long) $= 47{,}760$ lb $= 47.76$ kips

Taking each of these forces times its respective moment arm yields:

OVERTURNING MOMENT FOR EACH ZONE:

21.35 kips $\times$ 15 ft/2 $= 160.125$ ftk
7.69 kips $\times$ (15 ft + 5 ft/2) $= 134.57$ ftk
8.27 kips $\times$ (20 ft + 5 ft/2) $= 186.07$ ftk
8.72 kips $\times$ (25 ft + 5 ft/2) $= 239.80$ ftk
19.28 kips $\times$ (30 ft + 10 ft/2) $= 674.80$ ftk
43.62 kips $\times$ (40 ft + 20 ft/2) $= 2{,}181.00$ ftk
47.62 kips $\times$ (60 ft + 20 ft/2) $= 3{,}333.40$ ftk

Contributing to these horizontal overturning forces would be the uplift on the roof:

47.76 kips $\times$ (40 ft/2) $= 955.2$ ftk

This yields a total overturning moment of 7,864.97 ftk.

Comparing this overturning moment to the resisting moment, we can determine the factor of safety relative for overturning:

$$\frac{28{,}460 \text{ ftk}}{7{,}864.97 \text{ ftk}} = 3.61$$

This is fine; the normal expectation is that we would have a safety factor of at least 1.5. Although this is quite a bit higher than the 2.19 safety factor that could be obtained with the IBC methodology, this is partially due to the fact that the IBC uses a 90 mph [*145 km/h*] wind speed, which would be 20.8 psf/16.4 psf = 1.27 [*944 Pa/784 Pa = 1.27*], or a 27% increase in wind overturning forces. It's your choice: there is a significant difference in the amount of work, and for preliminary calculations the UBC procedure gives satisfactory data. *In any event, you need to obtain a qualified engineer to engage in sophisticated calculations such as these.*

parallel to the joist if the slope is steep enough (12:12) to create significant axial loads. On the other hand, if you were attempting to determine the total force which was attempting to overturn the structure or cause it to slide, the projected area method would be more direct. This method applies all coefficients to both positive and negative pressure on the vertical or horizontal projection of the entire building. It yields a force which acts on the vertical (or horizontal) plane of the basic elevation (or plan), dimensions of the building.

Now that we've completed an overturning problem and understand how the principle works, we can better understand the structural implications of two prominent towers.

You have probably never thought about the differences in the shapes of the Eiffel Tower and the Campanile at Piazza San Marco in Venice. The Eiffel Tower is a steel open frame structure 986 ft [*300 m*] tall, while the Campanile is a masonry solid rectangular structure 325 ft [*99 m*] tall. The Eiffel Tower has a tapered configuration, which reduces the amount of surface area exposed to the higher wind pressures associated with height; this tapered configuration also allows the tower to have a wider stance to provide resistance to overturning. The Campanile, in contrast, has a uniform configuration over its entire height and a very narrow footprint. The Eiffel Tower, although made of a heavier material (steel weighs 492 pcf [*7,872 kg/m³*] and stone about 120 pcf [*1,920 kg/m³*]), is an open trusswork and is actually a very light construction system.

The weight (times the distance from the edge of the base) of these two structures provides the resistance to overturning which is illustrated in Example 1.2. Now comes the interesting part. If you look at the pressure coefficients C_q for these two conditions, you will note that for the Campanile, a solid rectangular tower, the coefficient is 1.4 in any direction, while for the Eiffel Tower, an open trussed framework, the highest coefficient is 4.0. Remember, the Eiffel Tower weighs less relative to the volume that it expresses and consequently has less resistance to overturning. Since the effect of wind forces due to the pressure coefficient is almost three times as great as that of the Campanile, we have a potential recipe for disaster. How did Eiffel resolve this dilemma?

He widened the base to provide a greater moment arm for the dead load that was resisting overturning. The heavy, solid Campanile can have a small footprint, while the light, open Eiffel Tower must make other accommodations.

The form of the Eiffel Tower is probably at least partially the result of aesthetic judgment; equally likely, it is the result of Eiffel's not wanting the tower to find itself in the Seine on a windy day. The visual character of architecture is often the expression of a structural principle. End of sermon.

1.5 LATERAL LOADS: EARTHQUAKE

Earthquake forces on structures result from movement of the ground on which the building is supported. The ground moved both vertically and horizontally in a wave-like motion, but as we mentioned in Section 1.1, the vertical components are typically neglected since structures are designed for code-specified vertical live loads and construction dead loads, although this combination is one of the loading conditions that Load and Resistance Factor Design (LRFD) considers. The horizontal dynamic forces are of more concern, because the oscillation of the building masses can amplify the effect of these seismic forces much like a child causing a swing to move by shifting his or her weight to reinforce the movement of the swing (see the discussion of the Tuned Pendulum Systems in Section 1.4.1). This is particularly true of very flexible structures subjected to several cycles of lateral movement of the ground and its subsequent rebound.

In order to understand how earthquake forces act on buildings, we can start by studying a simplified model of a building connected to the ground. If the acceleration of the ground due to the earthquake is a, the corresponding inertial force acting on the mass is $F = ma$ (Newton's law). The base of the building would have to resist this horizontal shear force and the other effects due to the point of application of this force, as shown in Fig. 1.11.

Rarely do we find a building that reacts as a perfectly rigid mass; all structures and materials are elastic. The effect of an earthquake on a building is very similar to what a person standing on a bus experiences when the bus starts with a sudden acceleration. Since the center of the body mass is located well above the floor, the inertial force will push the body backward, while the feet tend to remain in place. The person will instinctively step back to maintain his or her balance. For a building connected to the ground, this is not possible; it would be like a person whose feet are glued to the floor of the bus, and the consequences of a sudden acceleration or stop would be very painful.

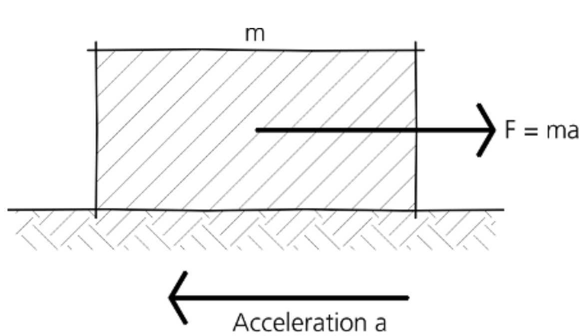


Fig. 1.11 Point of application of seismic force

Since most buildings, somewhat like the human body, are flexible and have no straps to hang on to, we can model this effect with a mass connected to the ground by a spring or even a person riding a horse sitting lightly in the saddle. This represents the effect of dead loads supported on elastic columns (Fig. 1.12).

If you pull the mass extending the spring, or bending an elastic stick, and release it, it will start oscillating with a type of movement mathematically similar to a pendulum subject to the force of gravity. The oscillation of the model is independent of the initial pull and depends on the elastic characteristics of the spring. The time it takes to complete a single oscillation, back and forth, is called the "period of oscillation"—T.

The acceleration of the ground (corresponding to the worst earthquake expected) is basically included in coefficients S_s (maximum ground motion: short period) or S_1 (maximum ground motion: 1 second)—Figs. 1.13 and 1.14, respectively.

Looking at these maps, it's easy to see where the areas of maximum earthquake damage might logically be expected to occur. In some areas on the West Coast, Fig. 1.14 shows the extreme variation in acceleration that can be expected around major fault lines.

If a series of pendulums is moved through the same ground motion that occurred in any given earthquake recorded by a seismograph, the maximum response of each pendulum can be recorded. The response may be deflection, velocity, acceleration, or shear, since they are all related. The response curve has very similar shapes for different earthquakes, although the magnitude of the response varies with the magnitude of the earthquake.

The actual magnitude of the earthquake is shown in the recordings of the acceleration (expressed as a percentage of the acceleration of gravity, g = 32.2 ft/sec/sec [*9.81 m/sec²*]) of the ground.

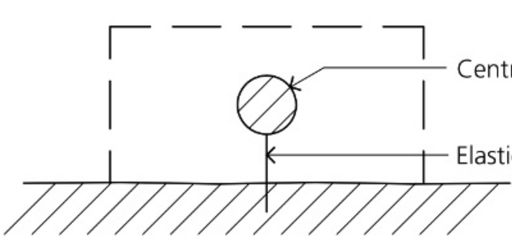


Fig. 1.12 Analogy of mass on an elastic support

It is very important, therefore, to know the seismic history of an area in order to determine the maximum acceleration to be expected. While many considerations beyond those included in this book need to be addressed in competent seismic design, the following—complicated as it may seem to you—is a simplified version that allows you to calculate fundamental values and understand the basic issues relative to the more complicated seismic design procedures. Remember, there is no substitute for a qualified structural engineer.

1.5.1 Earthquake-Resistant Construction

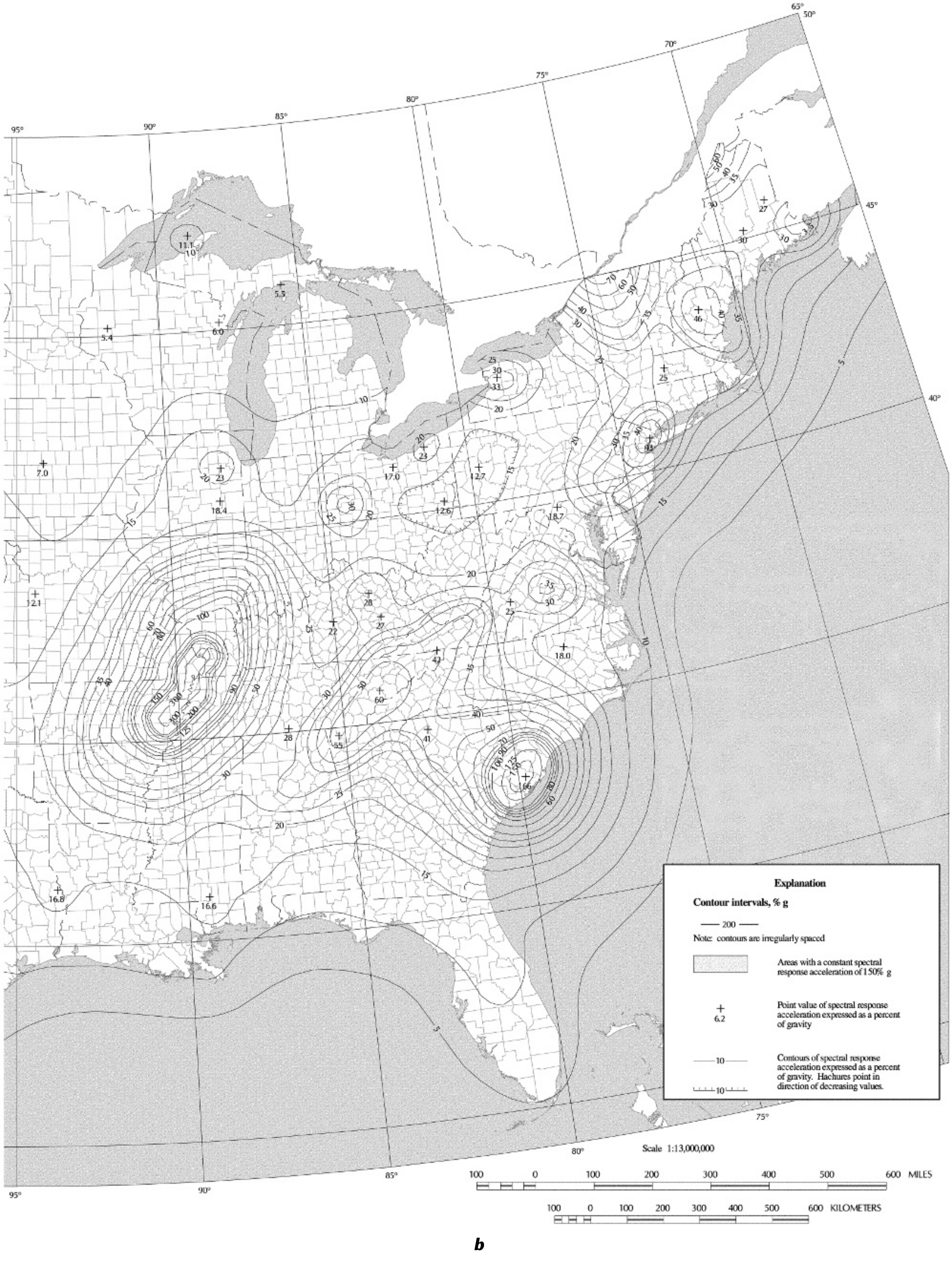
We have suggested that the vertical components of the structure will resist the loads in bending and/or shear. In small steel buildings, you can assume that the entire base shear is taken by the column bases, whereas in a small masonry building the shear is taken by the walls. This is a very simplified view, and a more accurate analysis should be made by a structural engineer. This approach essentially requires that we design the structural members to provide bending and shear resistance and to ensure that the connections will hold the pieces together (exterior wall panels must be adequately attached to the main building frame even if they have no structural function).

These potentially destructive forces must be acknowledged, and various strategies have been used that are analogous to systems we use in everyday life. If you're ever shipped anything, you know that you must pack items in a manner that will address the sometimes "rough handling" that the package will experience during shipment. You typically use some sort of energy absorbing material: crumpled newspapers, styrofoam peanuts, or bubble wrap. These are "dampening" devices that absorb the energy directed to the contents. So "damping" is a common technique. Damper elements can be installed as a retrofit in place of or in addition to normal diagonal bracing, a technique common in California and Japan.

A different strategy, which has implications for overall building design, is "seismic isolation." This technique consists of separating the building superstructure from the foundations to allow vertical loads to be transferred to the ground, while ground movements cannot be transmitted to the superstructure.

a

Figs. 1.13a, 1.13b Response acceleration at short periods S_S


Explanation
Contour intervals, % g
200
Note: contours are irregularly spaced
Areas with a constant spectral response acceleration of 150% g
Point value of spectral response acceleration expressed as a percent of gravity
6.2
Contours of spectral response acceleration expressed as a percent of gravity. Hachures point in direction of decreasing values.
Scale 1:13,000,000
100 0 100 200 300 400 500 600 MILES
100 0 100 200 300 400 500 600 KILOMETERS

b

Fig. 1.14 Los Angeles response acceleration at short period

Conceptually this is very simple, since you can imagine the column supported on sliding pads; if the foundations are shaken suddenly, the building remains still, held by its inertia (imagine a waiter pulling the tablecloth from under the dishes with a sudden jolt). Don't forget that an earthquake does not move the building, but only the ground to which the building is attached. If the building can slide on the ground, it will not experience, at least theoretically, any earthquake.

Of course, this floating movement must be restricted; consequently, the isolators are also damped and allow only limited movement (10 in. [*250 mm*]) and some transfer of lateral forces. An example of seismic isolators used in California consists of alternating layers of copolymers bonded to steel plates, separating the base of each column from the foundation, much like the motor mounts in your car that are intended to isolate engine vibrations from the passengers. They are similarly constructed of steel plates with a layer or layers of rubber between them to absorb and dampen vibrations.

In steel construction, the systems employed are braced frame systems or moment-resisting frames. Shear walls may be part of the system, often in the form of reinforced concrete elevator and stair shafts. The systems also can be used in combination—for example, a moment-resisting frame with a braced core or braced bents to resist the quake forces.

An alternative system, a Tuned Pendulum System, is being experimented with to compensate for the forces and reduce their effect on the structure. This is accomplished by suspending a large mass of material, typically water stored for fire fighting or mechanical systems. When a lateral force is applied to the structure, it moves sideways, but the pendulum swings out of phase with the building's movement. The pendulum is braced laterally by massive springs or shock absorbers which reduce its motion.

The Akashi-Kaikyo or "Pearl" Bridge in Japan, the longest suspension bridge in the world, four times as long as the Golden Gate bridge, uses a variation of the Tuned Pendulum System, referred to as Tuned Mass Dampers. This is the same fundamental idea used to control the lateral displacement of the two 928 ft [*283 m*] support towers during earthquakes. The system uses 20 of these dampers in each tower and was successfully, albeit unexpectedly, tested during construction in 1995 by a 7.2 Richter scale earthquake whose epicenter was directly under the bridge. While the towers permanently moved about 3 ft [*1 m*] from their original location, they withstood the forces without failure.

An example of this principle is the child in the swing previously discussed. If the child leans or shifts his weight properly, this will increase the amplitude of the swing, while if the timing is off, this same shifting of mass will cause no additional acceleration and may even stop the swing.

Several major concerns immediately come to mind when you put these issues together and the code identifies them as well. Buildings which have either vertical or horizontal discontinuities are subject to serious problems during an earthquake. Some of the most easily understood discontinuities include:

- *A nonuniform distribution of mass.* A clear example would be to place an apple on a long vertical metal rod. As you subject the rod to a horizontal force and move the apple to different locations, you'll immediately note that the movement is reduced or amplified, depending on the location of the apple relative to the base.
- *Sections of a building having different degrees of stiffness.* Using the steel rod from the previous example along with two other rods of different stiffnesses (diameter or cross sections) held together at the base, subject them to a sharp horizontal force. In this case, you'll see or can imagine that they move at different rates. Imagine what would happen if parts of a building were connected together. This differential in movement would immediately tear them apart.
- *Irregular geometries.* Consider an L-shaped building and imagine a horizontal force applied parallel to either leg. In one portion of the building, the length of the leg would resist the horizontal force, and in the other, the width of the leg would resist the force. Obviously, one is much stronger (stiffer) than the other; consequently, the point of connection between the legs would try to reconcile the two different magnitudes of movement. Result? Crash!

The 2003 IBC and the ASCE 7 elaborate on these basic ideas, but in general, irregularities in form or mass distribution are not good ideas. Think about the

Transamerica building in San Francisco. Is there a simple earthquake principle at work there?

1.5.2 Earthquake Design—Equivalent Lateral Force Procedure

The IBC defines the **basic structural systems,** for the purpose of earthquake design, for application of the R-factor. Within each of these categories there are several different options, so this is not really an architecturally limiting consideration. The options include:

1. Bearing walls
2. Building frames
3. Moment-resisting frames
4. Dual system with special moment frames
5. Dual systems with intermediate moment frames
6. Shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls
7. Inverted pendulum systems
8. Structural steel systems not specifically detailed for seismic resistance

In arriving at the seismic factor, the code takes into account the probability of a large earthquake, the type of building occupancy, the flexibility of the structure, and the physical site characteristics. Using symbols to represent these considerations, the seismic factor in its most simplified version is given by the following formula. This is known as the "equivalent lateral force procedure":

$$V_b = C_s W_s$$

where:

V_b = total base shear developed in kips

$C_s = \dfrac{S_{DS}}{R/I_E}$ (seismic response coefficient)

S_{DS} = design elastic response acceleration at a short period, $S_{DS} = \dfrac{2}{3} S_{MS}$

$S_{MS} = F_A S_S$

Table 1.10 Site Class Definitions

		Average Properties in Top 100 ft		
Site Class	**Soil Profile**	**Soil Shear Wave Velocity, $\bar{V}_s$ ft/s**	**Standard Penetration resistance $\bar{N}$**	**Soil Undrained Shear Strength, s_{ui} psf**
A	Hard rock	$\bar{V}_s \geq 5{,}000$	N/A	N/A
B	Rock	$2{,}500 \leq \bar{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil/soft rock	$1{,}200 \leq \bar{v}_s \leq 2{,}500$	$\bar{N} \geq 50$	$\bar{s}_{ui} \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_{ui} \leq 2{,}000$
E	Soil	$600 \leq \bar{v}_s \leq 1{,}200$	$\bar{N} \leq 15$	$\bar{s}_{ui} \leq 1{,}000$
		Any profile with more than 10 ft of soil having the following characteristics: 1. Plasticity index $PI \geq 20$ 2. Moisture content $w \geq 40$ 3. Undrained shear strength $\bar{s}_{ui} \leq 500$ psf		
F	Soils requiring site-specific evaluation	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible, weakly cemented soils. 2. Peats and/or highly organic clays ($H \geq 10$ ft of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H \geq 25$ ft with plasticity index $PI \geq 75$)		

For SI: 1 ft = 304.8 mm, 1 ft^2 = 0.0929 m^2, 1 lb/ft^2 = 0.0479 kPa, 1 ft/s = 0.305 m/s
N/A = not applicable.

F_A is a function of site classification and location, which determines S_S, and is listed in Table 1.11.

R = response modification factor from Tables 1.14a–1.14h

I_E = the importance factor from Table 1.7

W_S = effective "seismic" weight of the building in kips, consisting of the dead load and the following proportions of live loads:

1. In storage areas, 25% of the live load with any applicable reduction factors
2. In office structures or others that have a partition load, the actual partition weight or a minimum of 10 psf [*0.48 kN/m²*], whichever is greater
3. The total weight of any permanent equipment—mechanical equipment, fire suppression, water storage, and so on
4. Twenty percent of the flat roof snow load in areas where the ground snow load exceeds 30 psf [*1.44 kN/m²*]

If we substituted some of these factors into the original formula, we would get something that is a bit more recognizable in terms of the relevant factors:

$$V_b = \frac{S_{DS}}{R/I_E} W_s$$

This factor acknowledges the location and the expected seismic response (acceleration) of that location, the structural system, the importance of the facility, and the seismic weight of the entire building, all factors that you would expect to have an effect on the base shear.

This may seem pretty complicated and at first glance it is, but with a step-by-step methodical approach, it's not too bad. We'll do that after we discuss a few other items.

$$C_{s(max)} = \frac{S_{D1}}{T(R/I_E)}$$

$$C_{s(min)} = 0.044 S_{DS} I_E$$

S_{D1} = design elastic response acceleration at a 1.0-second period, $S_{D1} = \frac{2}{3} S_{M1}$

$$S_{M1} = F_V S_1$$

F_V may be found in Table 1.12 where you will see that it is also a function of the site classification and location which determine S_S and is listed in Fig. 1.13a and Fig. 1.13b.

S_1 is found in Figure 1.14a and 1.14b.

T is the fundamental period of the structure and may be established by "a properly substantiated analysis," as the IBC states. An alternative to this undefined criterion is to use $T_A = T$.

$$T_A = C_T h_N^X$$

C_T is a coefficient that may be determined from Table 1.13.

Table 1.11 F_A as a Function of Site Class and Short Period Acceleration S_S

	Mapped Maximum Considered Earthquake Spectral Response Acceleration at Short Periods				
Site Class	$S_S{}^b \le 0.25$	$S_S{}^b = 0.5$	$S_S{}^b = 0.75$	$S_S{}^b = 1.0$	$S_S{}^b \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	1[a]	1[a]	1[a]	1[a]	1[a]

[a] Site-specific geotechnical investigation and dynamic site response analyses shall be performed, except that for structures with periods of vibration equal to or less than 0.5 second, F_A values for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquification.

[b] Use straight line interpolation for intermediate values of S_S.

Table 1.12 F_V Values as a Function of Site Class and Mapped 1-Second Maximum Acceleration S_1

	Mapped Maximum Considered Earthquake Spectral Response Acceleration at 1-Second Periods				
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	[a]	[a]	[a]	[a]	[a]

[a] Site-specific geotechnical investigation and dynamic site response analyses shall be performed, except that for structures with periods of vibration equal to or less than 0.5 second, F_v values for liquefiable soils may be assumed equal to the values for the site class determined without regard to liquification.

[b] Use straight line interpolation for intermediate values of S_1.

h_N is the height above the base of the highest level of the building.

x is a coefficient that may be found in Table 1.13.

Alternatively, if the building does not exceed 12 stories in height (with a maximum floor-to-floor height of 10 ft [*3.05 m*]) and the lateral force resistance system consists entirely of concrete or steel moment resisting frames,

$$T_A = 0.1\ N$$

where N is the number of stories of the building.

All values in this formula have been previously defined and require only a substitution into this formula to double-check that you've met the minimum criteria.

Bearing walls and shear walls and their anchorage shall be designed for an out-of-plane force, F_p, that is the greater of the 10% of the weight of the wall or the quantity given by

$$F_p = 0.40\ I_E S_{DS} W_W$$

where:

I_E = occupancy importance factor (Table 1.5)

S_{DS} = short-period site design spectral response acceleration coefficient

W_W = weight of the wall

Table 1.13 Values of Approximate Period Parameters C_T and x

Structure Type	C_T	**x**
Moment-resisting frame systems of steel in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces.	0.028 [*0.068*]	0.8
Moment-resisting frame systems of reinforced concrete in which the frames resist 100% of the required seismic force and are not enclosed or adjoined by more rigid components that will prevent the frames from deflecting when subjected to seismic forces.	0.016 [*0.044*]	0.9
Eccentrically braced steel frames	0.03 [*0.07*]	0.75
All other structural systems	0.02 [*0.055*]	0.75

Italicized values are metric.

All parts of the structure, except at separation joints, shall be interconnected, and the connections shall be designed to resist the seismic force, F_p, induced by connection of the parts. Any smaller portion of the structure shall be tied to the remainder of the structure for the greater of

$$F_p = 0.133\ S_{DS}\ W_P \quad \text{or} \quad F_p = 0.05\ W_P$$

where:

S_{DS} = the design, 5% damped, spectral response acceleration at short periods (Figs. 1.13a and 1.13b)

W_P = the weight of the smaller portion

A positive connection for resisting a horizontal force acting parallel to the member shall be provided for each beam, girder, or truss to its support for a force not less than 5% of the dead plus live load.

1.5.3 Base Shear Force Distribution

The base shear V_B that we can calculate with the previous equations is distributed within the structure based on a proportioning system as follows:

$$F_X = C_{VX} V_B$$

F_X = the force at any level X, is

$$C_{vx} = \frac{w_x h^k{}_x}{\sum wh^k} = \text{the vertical distribution factor}$$

where:

w_X = the weight at level X

$h^k{}_X$ = the height of level X above the base

k = an exponent with the following definitions:

1. For structures having a period T_A of 0.5 second or less, k = 1.0
2. For structures having a period T_A of 2.5 seconds or more, k = 2.0

Table 1.14a Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Seismic-Force-Resisting System	R	System Height Limits (ft)				
1. Bearing wall systems		A/B	C	D	E	F
A. Ordinary steel-braced frames in light-frame construction	4.0	NL	NL	65	65	65
B. Special reinforced concrete shear walls	5.5	NL	NL	160	160	100
C. Ordinary reinforced concrete shear walls	4.5	NL	NL	NP	NP	NP
D. Detailed plain concrete shear walls	2.5	NL	NP	NP	NP	NP
E. Ordinary plain concrete shear walls	1.5	NL	NP	NP	NP	NP
F. Special reinforced masonry shear walls	5.0	NL	NL	160	160	100
G. Intermediate reinforced masonry shear walls	3.5	NL	NL	NP	NP	NP
H. Ordinary reinforced masonry shear walls	2.5	NL	160	NP	NP	NP
I. Detailed plain masonry shear walls	2.0	NL	NP	NP	NP	NP
J. Ordinary plain masonry shear walls	1.5	NL	NP	NP	NP	NP
K. Light frame walls with shear panels—wood structural panels or sheet steel panels	6.5	NL	NL	65	65	5.0
L. Light framed walls with shear panels—all other materials	2.0	NL	NL	35	NP	NP
M. Ordinary plain prestressed masonry shear walls	1.5	NL	NP	NP	NP	NP
N. Intermediate prestressed masonry shear walls	2.5	NL	35	NP	NP	NP
O. Special prestressed masonry shear walls	4.5	NL	35	35	35	35

For SI: 1 ft = 304.8 mm, 1 lb/ft^2 = 0.0479 kN/m^2.

Table 1.14b Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Seismic-Force-Resisting System	R	System Height Limits (ft)				
2. Building frame systems		A/B	C	D	E	F
A. Steel eccentrically braced frames, moment-resisting, connections at columns away from links	8.0	NL	NL	160	160	100
B. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7.0	NL	NL	160	160	100
C. Special steel concentrically braced farmes	6.0	NL	NL	160	160	100
D. Ordinary steel concentrically braced frames	5.0	NL	NL	35[a]	35[a]	NP[a]
E. Special reinforced concrete shear walls	6.0	NL	NL	160	160	100
F. Ordinary reinforced concrete shear walls	5.0	NL	NL	NP	NP	NP
G. Detailed plain concrete shear walls	3.0	NL	NP	NP	NP	NP
H. Ordinary plain concrete shear walls	2.0	NL	NP	NP	NP	NP
I. Composite eccentrically braced frames	8.0	NL	NL	160	160	100
J. Composite concentrically braced frames	5.0	NL	NL	160	160	100
K. Ordinary composite braced frames	3.0	NL	NL	NP	NP	NP
L. Composite steel place shear walls	6.5	NL	NL	160	160	100
M. Special composite reinforced concrete shear walls with steel elements	6.0	NL	NL	160	160	100
N. Ordinary composite reinforced concrete shear walls with steel elements	5.0	NL	NL	NP	NP	NP
O. Special reinforced masonry shear walls	5.5	NL	NL	160	160	100
P. Intermediate reinforced masonry shear walls	4.0	NL	NL	NP	NP	NP
Q. Ordinary reinforced masonry shear walls	3.0	NL	160	NP	NP	NP
R. Detailed plain masonry shear walls	2.5	NL	NP	NP	NP	NP
S. Ordinary plain masonry shear walls	1.5	NL	NP	NP	NP	NP
T. Light frame walls with shear panels—wood structural panels or sheet steel panels	7.0	NL	NL	65	65	65
U. Light framed walls with shear panels—all other materials	2.5	NL	NL	35	NP	NP
V. Ordinary plain prestressed masonry shear walls	1.5	NL	NP	NP	NP	NP
W. Intermediate prestressed masonry shear walls	3.0	NL	35	NP	NP	NP
X. Special prestressed masonry shear walls	4.5	NL	35	35	35	35

For SI: 1 ft = 304.8 mm; 1 lb/ft^2 = 0.0479 kN/m^2.

Table 1.14c Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Seismic-Force-Resisting System	R	System Height Limits (ft)				
3. Moment-resisting frame systems		A/B	C	D	E	F
A. Special steel moment frames	8.0	NL	NL	NL	NL	NL
B. Special steel truss moment frames	7.0	NL	NL	160	100	NP
C. Intermediate steel moment frames	4.5	NL	NL	35[b]	NP[b,c]	NP[b,c]
D. Ordinary steel moment frames	3.5	NL	NL	NP[b,c]	NP[b,c]	NP[b,c]
E. Special reinforced concrete moment frames	8.0	NL	NL	NL	NL	NL
F. Intermediate reinforced concrete moment frames	5.0	NL	NL	NP	NP	NP
G. Ordinary reinforced concrete moment frames	3.0	NL	NP	NP	NP	NP
H. Special composite moment frames	8.0	NL	NL	NL	NL	NL
I. Intermediate composite moment frames	5.0	NL	NL	NP	NP	NP
J. Composite partially restrained moment	6.0	160	160	100	NP	NP
K. Ordinary composite moment frames	3.0	NL	NP	NP	NP	NP
L. Masonry wall frames	5.5	NL	NL	160	160	100

For SI: 1 ft = 304.8 mm, 1 lb/ft^2 = 0.0479 kN/m^2.

Table 1.14d Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Seismic-Force-Resisting System	R	System Height Limits (ft)				
4. Dual systems with special moment frames		A/B	C	D	E	F
A. Steel eccentrically braced frames, moment-resisting connections, at columns away from links	8.0	NL	NL	NL	NL	NL
B. Steel eccentrically braced frames, non-moment-resisting, connections at columns away from links	7.0	NL	NL	NL	NL	NL
C. Special steel concentrically braced frames	8.0	NL	NL	NL	NL	NL
D. Special reinforced concrete shear walls	8.0	NL	NL	NL	NL	NL
E. Ordinary reinforced concrete shear walls	7.0	NL	NL	NP	NP	NP
F. Composite eccentrically braced frames	8.0	NL	NL	NL	NL	NL
G. Composite concentrically braced frames	6.0	NL	NL	NL	NL	NL
H. Composite steel plate shear walls	8.0	NL	NL	NL	NL	NL
I. Special composite reinforced concrete shear walls with steel elements	8.0	NL	NL	NL	NL	NL
J. Ordinary composite reinforced concrete shear walls with steel elements	7.0	NL	NL	NP	NP	NP
K. Special reinforced masonry shear walls	7.0	NL	NL	NL	NL	NL
L. Intermediate reinforced masonry shear walls	6.5	NL	NL	NP	NP	NP

For SI: 1 ft = 304.8 mm, 1 lb/ft^2 = 0.0479 kN/m^2.

Table 1.14e Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Seismic-Force-Resisting System	R	System Height Limits (ft)				
5. Dual systems with intermediate moment frames[d]	R	A/B	C	D	E	F
A. Special steel concentrically braced frames[e]	4.5	NL	NL	35	NP	NP
B. Special reinforced concrete shear walls	6.0	NL	NL	160	100	100
C. Ordinary reinforced concrete shear walls	5.5	NL	NL	NP	NP	NP
D. Ordinary reinforced masonry shear walls	3.0	NL	160	NP	NP	NP
E. Intermediate reinforced masonry shear walls	5.0	NL	NL	NP	NP	NP
F. Composite concentrically braced frames	3.0	NL	NL	160	100	NP
G. Ordinary composite braced frames	4.0	NL	NL	NP	NP	NP
H. Ordinary composite reinforced concrete shear walls with steel elements	5.5	NL	NL	NP	NP	NP

For SI: 1 ft = 304.8 mm, 1 lb/ft^2 = 0.0479 kN/m^2.

Table 1.14f Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Seismic-Force-Resisting System	R	System Height Limits (ft)				
		A/B	C	D	E	F
6. Shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls.	5.5	NL	NP	NP	NP	NP

For SI: 1 ft = 304.8 mm, 1 lb/ft^2 = 0.0479 kN/m^2.

Table 1.14g Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Seismic-Force-Resisting System	R	System Height Limits (ft)				
7. Inverted pendulum systems		A/B	C	D	E	F
A. Cantilevered column systems	2.5	NL	NL	35	35	35
B. Special steel moment frames	2.5	NL	NL	NL	NL	NL
C. Ordinary steel moment frames	1.25	NL	NL	NP	NP	NP
D. Special reinforced concrete moment frames	2.5	NL	NL	NL	NL	NL

For SI: 1 ft = 304.8 mm, 1 lb/ft^2 = 0.0479 kN/m^2.

Table 1.14h Design Coefficients and Factors for Basic Seismic-Force-Resisting Systems

Seismic-Force-Resisting System	R	System Height Limits (ft)				
		A/B	C	D	E	F
8. Structural steel systems not specifically detailed for seismic resistance.	3.0	NL	NL	NP	NP	NP

For SI: 1 ft = 304.8 mm, 1 lb/ft^2 = 0.0479 kN/m^2. NL = not limited; NP = not permitted.

[a] See IBC Section 1617.6.2.4.1 for a description of building systems limited to buildings with a height of 240 ft [*73.2 m*] or less. See IBC Section 1617.6.2.4.1 for building systems limited to buildings with a height of 160 ft [*48.8 m*] or less.

[b] Steel ordinary moment frames and intermediate moment frames are permitted in single-story buildings up to a height of 60 ft [*18.3 m*] when the moment joints of field connections are constructed of bolted end plates and the dead load of the roof does not exceed 15 psf [*717 N/m²*]. The dead weight of the portion of walls more than 35 ft [*107 m*] above the base shall not exceed 15 psf [*717 N/m²*].

[c] Steel ordinary moment frames are permitted in buildings up to a height of 35 ft [*10.7 m*], where the dead load of the walls, floors, and roof does not exceed 15 psf [*717 N/m²*].

[d] Steel intermediate moment resisting frames as part of a dual system are not permitted in Seismic Design Categories D, E, and F.

[e] An ordinary moment frame is permitted to be used in lieu of an intermediate moment frame in Seismic Design Categories B and C.

3. For structures having a period T_A between 0.5 and 2.5 seconds, k may be taken as 2.0 or as the linear interpolation between 1.0 and 2.0

$T_A = C_t h_n^x$, the approximate fundamental period of oscillation

V_B = the calculated base shear

In reality, structures do not behave like perfectly elastic bodies, and the response is affected by factors that decrease the magnitude of the response. This **damping** effect (see Section 1.5.1) can be caused by structural elements that absorb the accumulated energy of the structure (deformation of partitions and nonbearing walls, other forms of bracing with dampers, or energy dissipators). The coefficients used take damping into consideration with the R factor.

1.6 LOADING CONDITIONS

It is a common misconception that if you use the maximum amount of load, the members will be designed for the maximum condition.

The removal of load from a beam cantilever (Fig. 1.15) will actually increase the amount of moment the beam will experience as well as maximize the deflection between the supports. At the same time, removal of load between the supports will increase cantilever deflection but will cause no increase in the maximum moment in the cantilever. To design this or any other bending member, *any and all loading conditions which will maximize the moment, shear, and deflection must be considered.*

No amount of overdesigning will appropriately compensate for inadequate consideration of the critical loading conditions.

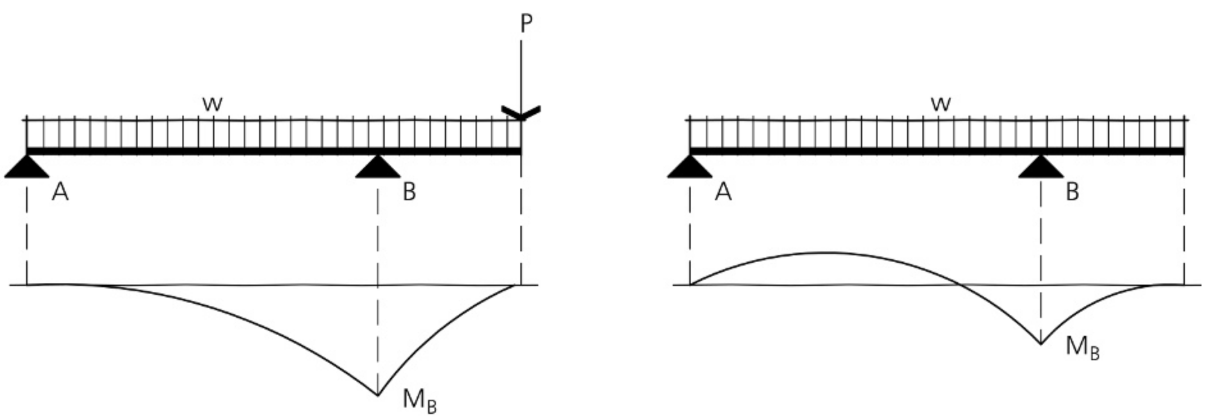


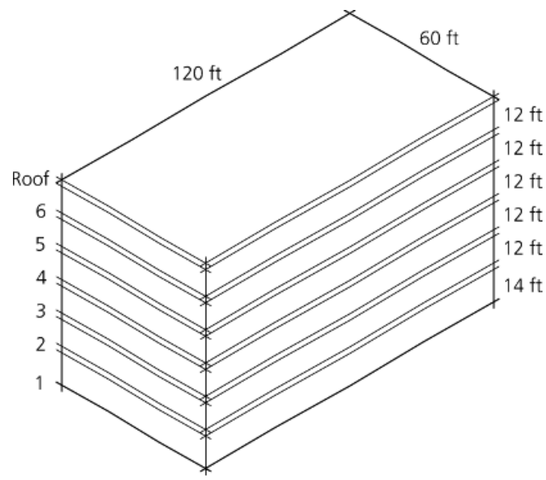
Fig. 1.15 Effect of load placement on a cantilevered beam

EXAMPLE 1.3

A 60×120 ft (height = 74 ft) six-story emergency dispatch and preparedness headquarters building in Indianapolis, Indiana, is constructed using a special steel concentrically braced frame that must be evaluated for base shear and for shear force distribution. The loading is illustrated in the diagram of the building.

The soil type in this area of the country is a very stiff soil overlying a relatively soft limestone at about 150 ft. This would qualify as a site D classification.

To determine the base shear, we will use the equivalent lateral force procedure and the following formula:



$$V_b = \frac{S_{DS}}{R/I_E} W_s$$

Let's take each part and work through it methodically.

1. V_b is the total base shear developed in kips, which is one of the pieces of information that we are requested to supply.

2. S_{DS} is the design elastic response acceleration at short period, $S_{DS} = \frac{2}{3} S_{MS}$, where

   $S_{MS} = F_A S_S$

   F_A is a function of site classification and location which determines S_S and is listed in Table 1.11. First, to determine F_A, we need to determine S_s, which is related to the site location and can be found on short period (0.5 second) acceleration map no. 1 Figs. 1.13a and 1.13b. This value is 0.2 for Indianapolis, Indiana. Note that the map values are "percentages" of the force of gravity and as such should be read as 0.20 for a value of 20.

   Since $S_{MS} = F_A S_S$, and since F_A equals 1.6 from Table 1.11 (site class D, $S_s \leq 0.25$), the value for $S_{MS} = 1.6(0.2) = 0.32$. Using this value, we can calculate the value for $S_{DS} = \frac{2}{3} S_{MS}$, $S_{DS} = \frac{2}{3} 0.32 = 0.22$.

3. R is the response modification factor from Tables 1.14a to 1.14h, in this case for a special steel concentrically braced frame; the value is equal to 6.0 from Table 1.14b. This is also a good time to verify that the building height does not exceed the maximum specified for this construction system in this site classification. Table 1.14b indicates that the building could have a maximum height of 160 ft. Our height of 74 ft falls well within the limit.

4. I_E is the importance factor from Table 1.7. This building would be a Category IV and would have an importance factor of 1.50.

5. W_S is the effective "seismic" weight of the building in kips, consisting of the dead load and the following proportions of live loads:

   a. In storage areas, 25% of the live load with any applicable reduction factors
   b. In office structures or others that have a partition load, the actual partition weight or a minimum of 10 psf [*0.48 kN/m²*], whichever is greater

c. The total weight of any permanent equipment—mechanical equipment, fire suppression, water storage, and so on

d. Twenty percent of the flat roof snow load in areas where the ground snow load exceeds 30 psf [*1.44 kN/m²*]

This one is a bit more complicated, but here we go:

For the floor loads, we have 35 psf DL, and using a minimum of 10 psf for a partition load (without additional specific information), we should design the floors for a seismic load of 45 psf. This would equal a total seismic load for each floor of 45 psf (60 ft) (120 ft) = 324,000 lb or 324 kips.

For the roof, we have 20 psf DL and 20 psf of snow load (since the snow load is less than 30 psf in Indianapolis, Indiana). This would equal a seismic load of 40 psf on the roof or a total seismic load of 40 psf (60 ft) (120 ft) = 288,000 lb or 288 kips.

So, the building weights for seismic calculations are:

Floor(s) = 324 k

Roof = 288 k

6. We now have all of the necessary values to calculate the base shear of the building using

$$V_b = \frac{S_{DS}}{R/I_E} W_s$$

$$= \frac{0.22}{6.0/1.50} (5(324 \text{ k}) + 288 \text{ k}) = 104.9 \text{ k}$$

7. To determine the force distribution throughout the building, we would use

$$F_X = C_{VX}V_B$$

where:

F_X = the force at any level *X*

$C_{vx} = \dfrac{w_x h^k{}_x}{\sum wh^k}$ = the vertical distribution factor

w_X = the weight at level X

$h^k{}_X$ = the height of level X above the base

k = an exponent with the following definitions:

1. For structures having a period T_A of 0.5 second or less, k = 1.0.
2. For structures having a period T_A of 2.5 seconds or more, k = 2.0.
3. For structures having a period T_A between 0.5 and 2.5 seconds, k may be taken as 2.0 or as the linear interpolation between 1.0 and 2.0.

$T_A = C_t h_n{}^x$, the approximate fundamental period of oscillation

V_B = the calculated base shear.

Again, we'll methodically calculate each term.

The first value we will need is the period of oscillation of the building: $T_A = C_t h_n^x$. From Table 1.13 we will use the value of 0.028 for C_t and 0.8 for x. This allows us to calculate $T_A = 0.028(74 \text{ ft})^{0.8} = 0.876$ second. Using a linear interpolation between 0.5 and 2.5 seconds, the value would be: at $T_A = 0.5$ second, k = 1.0; at $T_A = 2.5$ seconds, k = 2.0. Therefore, at $T_A = 0.876$ second, $k = 1.0 + \dfrac{0.876 - 0.5}{2.5 - 0.5}(1.0) = 1.188$, or conservatively we could use 2.0. We'll use the calculated value of 1.188, since we've already done it.

k = 1.188

8. It's time to create a small table of values for wh^k for each level and for the sum of the wh^k values.

For the 14 ft level: $wh^k = 324 \text{ k } (14 \text{ ft})^{1.188} = 7{,}449.8$ ftk

For the 26 ft level: $wh^k = 324 \text{ k } (26 \text{ ft})^{1.188} = 15{,}524.9$ ftk

For the 38 ft level: $wh^k = 324 \text{ k } (38 \text{ ft})^{1.188} = 24{,}396.5$ ftk

For the 50 ft level: $wh^k = 324 \text{ k } (50 \text{ ft})^{1.188} = 33{,}800.4$ ftk

For the 62 ft level: $wh^k = 324 \text{ k } (62 \text{ ft})^{1.188} = 43{,}642.2$ ftk

For the 74 ft level: $wh^k = 288 \text{ k } (74 \text{ ft})^{1.188} = 47{,}867.4$ ftk

The total of the wh^k values is $\sum wh^k = 172{,}699.2$ ftk

Now we can calculate the forces generated at each level:

For the 14 ft level: $F_{14} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{7{,}449.8 \text{ ftk}}{172{,}699.2 \text{ ftk}} 104.9\text{k} = 4.52 \text{ k}$

For the 26 ft level: $F_{26} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{15{,}542.9 \text{ ftk}}{172{,}699.2 \text{ ftk}} 104.9\text{k} = 9.40 \text{ k}$

For the 38 ft level: $F_{38} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{24{,}396.5 \text{ ftk}}{172{,}699.2 \text{ ftk}} 104.9\text{k} = 14.81 \text{ k}$

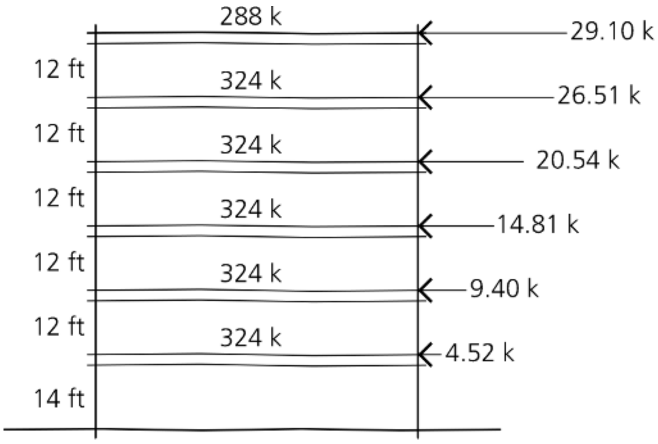
For the 50 ft level: $F_{50} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{33{,}800.4 \text{ ftk}}{172{,}699.2 \text{ ftk}} 104.9\text{k} = 20.54 \text{ k}$

For the 62 ft level: $F_{62} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{43{,}642.2 \text{ ftk}}{172{,}699.2 \text{ ftk}} 104.9\text{k} = 26.51 \text{ k}$

For the 74 ft level: $F_{74} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{47{,}867.4 \text{ ftk}}{172{,}699.2 \text{ ftk}} 104.9\text{k} = 29.10 \text{ k}$

We can check our answer, since the sum of the forces at all levels should equal 104.9 k and it equals 104.88 k; that's about as good as we can expect.

Well, it was lengthy, but not really hard.



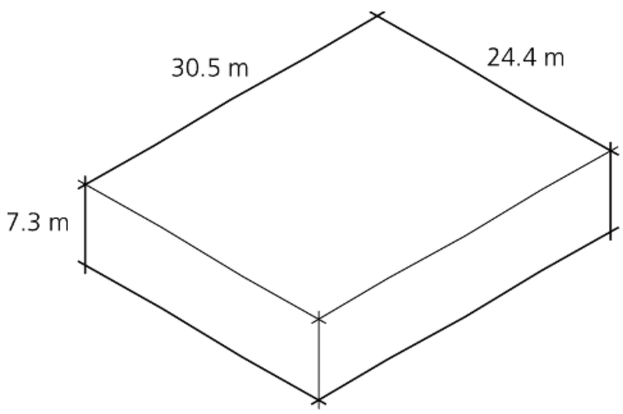
Metric Versions of All Examples and Problems

EXAMPLE M1.1

A school in a flat urban area, Indianapolis, Indiana, with a footprint of 24.4 m × 30.5 m with a wall height of 7.3 m, flat roof, Exposure B.

Using the UBC approach, determine the total horizontal and vertical forces acting on this building using the normal force method, which would be most appropriate for designing individual members or components of the structure.

$P = C_e C_q q_s I_w$



(a) Positive Pressure: Inward:

EXPOSURE COEFFICIENTS:

$C_e = 0.62$ 0–4.6 m (Table 1.6)
$C_e = 0.67$ 4.6–6.1 m
$C_e = 0.72$ 6.1–7.6 m

PRESSURE COEFFICIENT:

$C_q = 0.8$

Basic wind speed from Fig. 1.10 = 129 km/h; therefore, $q_s = 784\ N/m^2$.

$I_w = 1.0$ special occupancy structure (Table 1.9)

PRESSURE:

0–4.6 m $P_1 = 0.62 \times 0.8 \times 784\ N/m^2 \times 1.0 = 389\ N/m^2$
4.6–6.1 m $P_2 = 0.67 \times 0.8 \times 784\ N/m^2 \times 1.0 = 420\ N/m^2$
6.1–7.6 m $P_3 = 0.72 \times 0.8 \times 784\ N/m^2 \times 1.0 = 452\ N/m^2$

(b) Negative Pressure: Outward:

EXPOSURE COEFFICIENTS:

$C_e = 0.62$ 0–4.6 m (Table 1.14)
$C_e = 0.67$ 4.6–6.1 m
$C_e = 0.72$ 6.1–7.6 m

PRESSURE COEFFICIENT:

$C_q = 0.5$

PRESSURE:

0–4.6 m $P_1 = 0.62 \times 0.5 \times 784\ N/m^2 \times 1.0 = 243\ N/m^2$
4.6–6.1 m $P_2 = 0.67 \times 0.5 \times 784\ N/m^2 \times 1.0 = 263\ N/m^2$
6.1–7.6 m $P_3 = 0.72 \times 0.5 \times 784\ N/m^2 \times 1.0 = 282\ N/m^2$

(c) Roof Uplift: Outward:

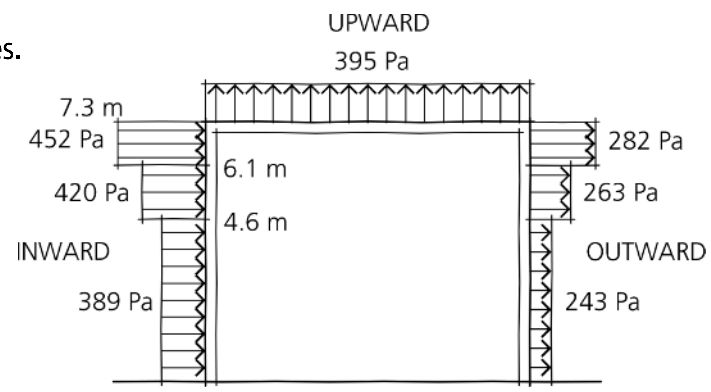
$C_e = 0.72$ (based on mean roof height = 7.3 m, which puts us in the 6.1–7.6 m range)

$C_q = 0.7$

$P_4 = 0.72 \times 0.7 \times 784\ N/m^2 \times 1.0 = 395\ N/m^2$

These pressures must be applied simultaneously perpendicular to the walls and to the roof surfaces. The vertical loads can be considered to resist overturning and suction.

Note that these pressures are lower than the ones obtained using the IBC procedures. This is because the IBC uses a 145 km/h wind classification for this region and the UBC uses 145 km/h. Additionally, the IBC uses a higher value at the end zones to account for the compression of the wind as it "rounds the corners," and this factor is not considered in the UBC versions.



EXAMPLE M1.2

We'll now work a different problem with a 24.4 m tall, slender office building which has a total weight (worst conditions with DL only) of 6,330 kN and a footprint of 12.2 × 30.5 m, again using the UBC approach but the projected area method. This method is best used when evaluating the overall overturning forces on the building, which will likely become an issue in a taller structure with a relatively narrow base.

The building weighs 6,330 kN, and with a 12.2 m base, assuming it will "topple" about the narrow dimension (a pretty clear option), it has an overturning **resisting moment** (weight × moment arm) of 6,330 kN (12.2 m/2) = 38,613 kNm. This is the force which would hold it in place under high wind loads.

The overturning forces can be found by calculating the overturning wind forces acting on the structure.

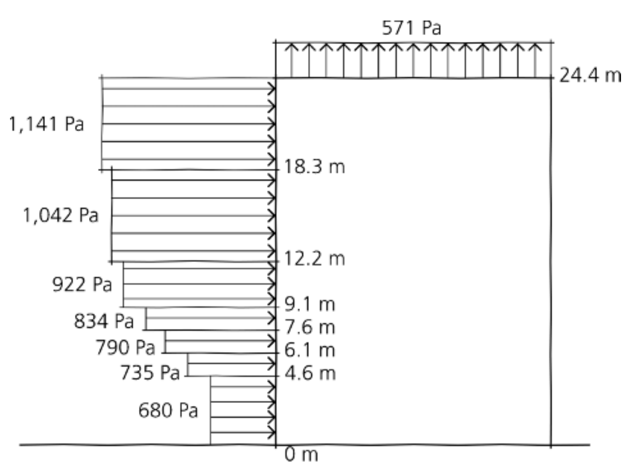
(a) Horizontal Pressure

EXPOSURE COEFFICIENTS:

$C_e = 0.62$ 0–4.6 m (Table 1.6)
$C_e = 0.67$ 4.6–6.1 m
$C_e = 0.72$ 6.1–7.6 m
$C_e = 0.76$ 7.6–9.1 m
$C_e = 0.84$ 9.1–12.2 m
$C_e = 0.95$ 12.2–18.3 m
$C_e = 1.04$ 18.3–24.4 m

PRESSURE COEFFICIENT:

$C_q = 1.4$



(b) Total Gross Pressure (Inward + Outward)

Basic wind speed from Fig. 1.10 is 129 km/h; therefore, $q_s = 784$ N/m^2.

$I_w = 1.0$ primary frames and systems (Table 1.9)

PRESSURE:

0–4.6 m	$P_1 = 0.62 \times 1.4 \times 784$ N/m$^2 \times 1.0 = 680$ N/m^2
4.6–6.1 m	$P_2 = 0.67 \times 1.4 \times 784$ N/m$^2 \times 1.0 = 735$ N/m^2
6.1–7.6 m	$P_3 = 0.72 \times 1.4 \times 784$ N/m$^2 \times 1.0 = 790$ N/m^2
7.6–9.1 m	$P_4 = 0.76 \times 1.4 \times 784$ N/m$^2 \times 1.0 = 834$ N/m^2
9.1–12.2 m	$P_5 = 0.84 \times 1.4 \times 784$ N/m$^2 \times 1.0 = 922$ N/m^2
12.2–18.3 m	$P_6 = 0.95 \times 1.4 \times 784$ N/m$^2 \times 1.0 = 1,042$ N/m^2
18.3–24.4 m	$P_7 = 1.04 \times 1.4 \times 784$ N/m$^2 \times 1.0 = 1,141$ N/m^2

If you take the values for positive and negative pressure acting on opposite sides of the building from the problem above, you will note that on the positive side the C_e factor is 0.8 and on the negative side it is 0.5, which total 1.3. This is the C_e factor for buildings under 12.2 m using the projected area method. For taller buildings, there is a difference since the C_e factor becomes 1.4, as we saw in this problem.

This suggests that if you want only the total horizontal forces, the projected area method gives you the same answers with 50% of the work. Take your choice.

(c) Roof Uplift

EXPOSURE COEFFICIENT:

$C_e = 1.04$ (using a roof height of 24.4 m)

PRESSURE COEFFICIENT:

$C_q = 0.7$

PRESSURE:

$P_{roof} = 1.04 \times 0.7 \times 784\ \text{N/m}^2 \times 1.0 = 571\ \text{N/m}^2$

To calculate the overturning moment, we determine the total force acting at the centroid of each level and multiply it by its moment arm relative to the base.

TOTAL WIND FORCE AT EACH ZONE:

0–4.6 m	$F_1 = 680\ \text{N/m}^2$	$\times$ (4.6 m high $\times$ 30.5 m long) $= 95{,}404$ N $= 95.40$ kN
4.6–6.1 m	$F_2 = 735\ \text{N/m}^2$	$\times$ (1.5 m high $\times$ 30.5 m long) $= 33{,}626$ N $= 33.63$ kN
6.1–7.6 m	$F_3 = 790\ \text{N/m}^2$	$\times$ (1.5 m high $\times$ 30.5 m long) $= 36{,}142$ N $= 36.14$ kN
7.6–9.1 m	$F_4 = 834\ \text{N/m}^2$	$\times$ (1.5 m high $\times$ 30.5 m long) $= 38{,}155$ N $= 38.15$ kN
9.1–12.2 m	$F_5 = 922\ \text{N/m}^2$	$\times$ (3.1 m high $\times$ 30.5 m long) $= 87{,}175$ N $= 87.17$ kN
12.2–18.3 m	$F_6 = 1{,}042\ \text{N/m}^2$	$\times$ (6.1 m high $\times$ 30.5 m long) $= 193{,}864$ N $= 193.86$ kN
18.3–24.4 m	$F_7 = 1{,}141\ \text{N/m}^2$	$\times$ (6.1 m high $\times$ 30.5 m long) $= 212{,}283$ N $= 212.28$ kN
	Rooftop uplift $= 571\ \text{N/m}^2$	$\times$ (12.2 m wide $\times$ 30.5 m long) $= 212{,}469$ N $= 212.47$ kN

Taking each of these forces times its respective moment arm yields:

OVERTURNING MOMENT FOR EACH ZONE:

$95.40\ \text{kN} \times 4.6\ \text{m}/2 = 219.42\ \text{kNm}$

$33.63\ \text{kN} \times (4.6\ \text{m} + 1.5\ \text{m}/2) = 179.92\ \text{kNm}$

$36.14\ \text{kN} \times (6.1\ \text{m} + 1.5\ \text{m}/2) = 247.56\ \text{kNm}$

$38.15\ \text{kN} \times (7.6\ \text{m} + 1.5\ \text{m}/2) = 318.55\ \text{kNm}$

$87.17\ \text{kN} \times (9.1\ \text{m} + 3.1\ \text{m}/2) = 928.36\ \text{kNm}$

$193.86\ \text{kN} \times (12.2\ \text{m} + 6.1\ \text{m}/2) = 2956.36\ \text{kNm}$

$212.28\ \text{kN} \times (18.3\ \text{m} + 6.1\ \text{m}/2) = 4532.18\ \text{kNm}$

Contributing to these horizontal overturning forces would be the uplift on the roof:

$$212.47 \text{ kN} \times (12.2 \text{ m}/2) = 1296.07 \text{ kNm}$$

This yields a total overturning moment of 10,678.42 kNm.

Comparing this overturning moment to the resisting moment, we can determine the safety factor for overturning: 38,613 kNm.

$$\frac{38{,}613 \text{ kNm}}{10{,}678 \text{ kNm}} = 3.61$$

This is fine; normally, we would expect to have a safety factor of at least 1.5. Although this is quite a bit higher than the 2.19 safety factor that could be obtained using the IBC methodology, this is partially due to the fact that the IBC used a 145 km/h wind speed, which would be 944 N/m^2/784 N/m^2 = 1.27, or a 27% increase in wind overturning forces. It's your choice: there is a significant difference in the amount of work, and for preliminary calculations the UBC procedure gives satisfactory data. *In any event, you need to obtain the services of a qualified engineer to engage in sophisticated calculations such as these.*

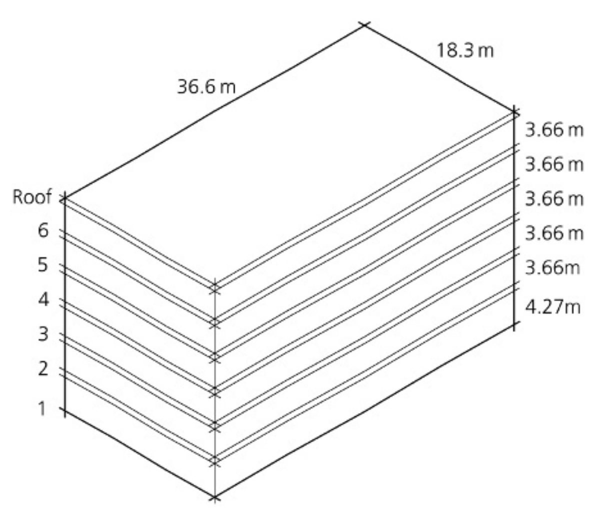
EXAMPLE M1.3

A 18.3 m × 36.6 m six-story (height = 22.6 m) emergency dispatch and preparedness headquarters building in Indianapolis, Indiana, constructed using a special steel concentrically braced frame must be evaluated for base shear and for shear force distribution. The loading is illustrated in the diagram of the building.

The soil type in this area of the country is a very stiff soil profile overlying a relatively soft limestone at about 45 m. This would qualify as a site D classification.

To determine the base shear, we will use the equivalent lateral force procedure and the formula

$$V_b = \frac{S_{DS}}{R/I_E} W_s$$



Let's take each part and work through it methodically.

1. **1.** V_b is the total base shear developed in kips; that's obviously one of the specific pieces of information that we are requested to supply.

2. **2.** S_{DS} is the design elastic response acceleration at short period, $S_{DS} = \frac{2}{3} S_{MS}$, where

   $S_{MS} = F_A S_S$.

   F_A is a function of site classification and location which determines S_S and is listed in Table 1.11. First, to determine F_A, we need to determine S_s, which is related to the site location and can be found on short period (0.5-second) acceleration map no. 1 (Figs. 1.13a and 1.13b). This value is 0.2 for Indianapolis. Note that the map values are percentages of the force of gravity and as such should be read as 0.20 for a value of 20.

   Since $S_{MS} = F_A S_S$, and since F_A equals 1.6 from Table 1.11 (site class D, $S_s \leq 0.25$), the value for $S_{MS} = 1.60(0.2) = 0.32$.

   Using this value, we can calculate the value for $S_{DS} = \frac{2}{3} S_{MS}$, $S_{DS} = \frac{2}{3} 0.32 = 0.22$.

3. **3.** R is the response modification factor from Tables 1.14a to 1.14h. In this case, for a special steel concentrically braced frame, the value is equal to 6.0 from Table 1.14b. This is also a good time to verify that the building height does not exceed the maximum specified for this construction system in this site classification. Table 1.14b indicates that the building could have a maximum height of 48.8 m. Our height of 22.6 m falls well within the limit.

4. **4.** I_E is the importance factor from Table 1.7. This building would be a Category IV and would have an importance factor of 1.50.

5. **5.** W_S is the effective "seismic" weight of the building in kips, consisting of the dead load and the following proportions of live loads:

   a. In storage areas, 25% of the live load with any applicable reduction factors
   b. In office structures or others that have a partition load, the actual partition weight or a minimum of 470 N/m^2, whichever is greater
   c. The total weight of any permanent equipment—mechanical equipment, fire suppression, water storage, and so on.
   d. Twenty percent of the flat roof snow load in areas where the ground snow load exceeds 1,440 N/m^2

   This one is a bit more complicated, but here we go:

   For the floor loads, we have 1,673 N/m^2 DL, and using a minimum of 470 N/m^2 for a partition load (without additional specific information), we should design the floors for a seismic load of 2,144 N/m^2. This would equal a total seismic load for each floor of 2,144 N/m^2 (18.3 m) (36.6 m) $=$ 1,436,008 N or 1,436 kN.

   For the roof, we have 942 N/m^2 DL and 956 N/m^2 of snow load (since the snow load is less than 1,440 N/m^2 in Indianapolis). This would equal a seismic load of 1,898 N/m^2 on the roof or a total seismic load of 1,898 N/m^2 (18.3 m) (36.6 m) $=$ 1,271,200 N or 1271.2 kN.

So, the building weight for seismic calculations are:

Floor(s) = 1,436 kN

Roof = 1,271.2 kN

6. We now have all of the necessary values to calculate the base shear of the building using

$$V_b = \frac{S_{DS}}{R/I_E} W_s \qquad V_b = \frac{0.22}{6.0/1.50}\,[5(1{,}436\text{ kN}) + 1{,}271.2\text{ kN}] = 464.8\text{ kN}$$

7. To determine the force distribution throughout the building, we would use

$$F_X = C_{VX} V_B$$

where:

F_X = the force at any level X

$C_{vx} = \dfrac{w_x h^k{}_x}{\sum wh^k}$ = the vertical distribution factor

w_X = the weight at level X

$h^k{}_X$ = the height of level X above the base

k = an exponent with the following definitions:

a. For structures having a period T_A of 0.5 second or less, k = 1.0.
b. For structures having a period T_A of 2.5 seconds or more, k = 2.0.
c. For structures having a period T_A between 0.5 and 2.5 seconds, k may be taken as 2.0 or as the linear interpolation between 1.0 and 2.0.

$T_A = C_t h_n{}^x$, approximate fundamental period of oscillation

V_B = the calculated base shear

Again, we'll methodically calculate each term.

The first value we will need is the period of oscillation of the building: $T_A = C_t h_n{}^x$. From Table 1.13, we will use the values of 0.068 for C_t and 0.8 for x. This allows us to calculate $T_A = 0.068(22.6\text{ m})^{0.8} = 0.823$ second. Using a linear interpolation between 0.5 and 2.5 seconds, the value would be: at T_A = 0.5 second, k = 1.0; at T_A = 2.5 seconds, k = 2.0. Therefore, at T_A = 0.823 second, $k = 1.0 + \dfrac{0.823 - 0.5}{2.5 - 0.5}(1.0) = 1.161$, or conservatively we could use 2.0. We'll use the calculated value of 1.161, since we've already done it.

k = 1.161

8. It's time to create a small table of values for wh^k for each level and for the sum of the wh^k values:

For the 4.27 m level: $wh^k = 1{,}436\text{ kN}\,(4.27\text{ m})^{1.161} = 7{,}746.6$ kNm

For the 7.93 m level: $wh^k = 1{,}436\text{ kN}\,(7.93\text{ m})^{1.161} = 15{,}894.3$ kNm

For the 11.59 m level: $wh^k = 1{,}436\text{ kN}\,(11.59\text{ m})^{1.161} = 24{,}693.7$ kNm

For the 15.25 m level: $wh^k = 1{,}436\text{ kN}\,(15.25\text{ m})^{1.161} = 33{,}959.5$ kNm

For the 18.91 m level: $wh^k = 1{,}436\ \text{kN}\ (18.91\ \text{m})^{1.161} = 43{,}593.7\ \text{kNm}$

For the 22.6 m level: $wh^k = 1{,}271.2\ \text{kN}\ (22.6\ \text{m})^{1.161} = 47{,}460.7\ \text{kNm}$

The total of the wh^k values is $\sum wh^k = 173{,}348.4\ \text{kNm}$.

Now we can calculate the forces generated at each level:

For the 4.27 m level: $F_{14} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{7{,}746.6\ \text{kNm}}{173{,}348.4\ \text{kNm}} 464.8\ \text{kN} = 20.77\ \text{kN}$

For the 7.93 m level: $F_{26} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{15{,}894.3}{173{,}348.4\ \text{kNm}} 464.8\ \text{kN} = 42.62\ \text{kN}$

For the 11.59 m level: $F_{38} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{24{,}693.7\ \text{kNm}}{173{,}348.4\ \text{kNm}} 464.8\ \text{kN} = 66.21\ \text{kN}$

For the 15.25 m level: $F_{50} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{33{,}959.5\ \text{kNm}}{173{,}348.4\ \text{kNm}} 464.8\ \text{kN} = 91.06\ \text{kN}$

For the 18.91 m level: $F_{62} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{43{,}593.7\ \text{kNm}}{173{,}348.4\ \text{kNm}} 464.8\ \text{kN} = 116.89\ \text{kN}$

For the 22.6 m level: $F_{74} = \dfrac{w_x h^k_x}{\sum wh^k} V_B = \dfrac{47{,}460.7\ \text{kNm}}{173{,}348.4\ \text{kNm}} 464.8\ \text{kN} = 127.26\ \text{kN}$

We can check our answer since the sum of the forces at all levels should equal 464.8 kN, and it equals 464.8 kN. That's about as good as we can expect.

Well, it was lengthy, but not really hard.