where:

p = design pressure in psf; positive value means acting toward the surface, negative value means acting away from the surface.
F = design force in pounds.
q = velocity pressure in psf; q_z is the value determined at height z (ft) above ground, and q_h is the value determined at mean roof height h (ft).
G = gust response factor (dimensionless); G_z is the value determined at height z (ft) above ground, and G_h is the value determined at mean roof height h (ft).
A = area of structure or cladding and component (sq ft).
C = pressure coefficient (dimensionless); positive value means acting toward the surface, whereas negative value means acting away from the surface; C_p is external pressure coefficient, C_{pi} is internal pressure coefficient, and C_f is force coefficient.

Design pressures and forces are determined for each structure separately for main wind-force resisting systems and for components and cladding. There are two reasons for this: (1) it is recognized that the spatial extent of wind gust may engulf components, but not the entire structure; and (2) gust response characteristics of a component would be significantly different from that of the whole structure.

The velocity pressure, q, in psf is given by:

$$q_z = 0.00256 K_z (IV)^2$$

where V is the basic wind speed in mph, I is the importance factor, K_z is the velocity exposure coefficient, and 0.00256 is a constant reflecting air mass density.

Basic wind speed, V, is defined as fastest-mile wind speed at 33 ft (10 m) above ground of terrain exposure C (flat open country and grassland) and associated with an annual probability of occurrence of 0.02. V for any location in the country can be determined from a contour map included in ANSI A58.1-82.

The importance coefficient, I, modifies wind speed to 100-year or 25-year mean recurrence intervals. It has a value (away from the hurricane ocean line) of 1.0 for usual structures, 1.07 for essential facilities and buildings for public assembly, and 0.95 for buildings that represent a low hazard to human life in the event of failure (e.g., agricultural buildings, minor storage facilities, etc.).

The velocity pressure exposure coefficient, K_z, takes into account changes in wind speed with height above ground and with the nature of the surroundings (types of terrain). It is recognized that the wind speed varies with height because of ground friction, and that the amount of friction varies with the ground roughness.

$$K_z = 2.58\left(\frac{z}{z_g}\right)^{2/d}$$

where:

z = elevation in feet.
z_g = gradient height in feet; at this height wind velocity becomes constant.
α = coefficient depending on exposure.

Four roughness categories or exposure conditions are considered:

1. Centers of large cities and very rough terrain, Exposure A ($z_g = 1500$ ft, $\alpha = 3$, $D_0 = 0.025$, where D_0 = surface drag coefficient, see below).
2. Suburban areas, towns, city outskirts wooded areas, and rolling terrain, Exposure B ($z_g = 1200$ ft, $\alpha = 4.5$, $D_0 = 0.010$).
3. Flat open country and grassland, Exposure C ($z_g = 900$ ft, $\alpha = 7$, $D_0 = 0.005$).
4. Flat, unobstructed coastal areas directly exposed to wind blowing over bodies of water, Exposure D ($z_g = 700$ ft, $\alpha = 10$, $D_0 = 0.003$).

The gust response factor, G, accounts for the additional loading effects due to wind turbulence over the fastest-mile speed of wind. It also includes loading effects due to dynamic amplification of flexible buildings and structures.

$$G_z = 0.65 + 3.65 T_z$$

where:

$$T_z = \frac{2.35(D_0)^{1/2}}{(z/30)^{1/\alpha}}$$

The pressure and force coefficients, C, for buildings and structures and their components and cladding are given in several figures and tables in ANSI A58.1-82.

The quasi-static approach to wind load design has generally proved sufficient for most structures. However, a more detailed analysis, including wind tunnel studies, may be appropriate for special structures. The above approach to wind may not be satisfactory for ultra-high-rise buildings, especially with respect to comfort of the occupants (in very flexible structures) and the permissible horizontal movement, or drift, which might result in cracking of partitions and glass. These important factors are related to the frequency and amplitude of the vibrations, which depend on the natural frequencies of the building and gust fluctuations of the wind, rather than on steady wind pressure.

10.5.2 Serviceability Criteria

With respect to wind design, the following aspects have to be considered to ensure the satisfactory performance of a structure under service conditions:

(a) Lateral deflection of the structure, particularly as this affects its stability and the cracking of nonstructural elements and structural members.
(b) Motion of the structure, as it affects comfort of the occupants.

1. *Lateral deflection* or drift is the magnitude of displacement at the top of a building relative to its base. The ratio of the total lateral deflection to the building height, or the story deflection to the story height, is referred to as the "deflection index." The imposition of a maximum allowable lateral sway (drift) is based on the need to limit the possible adverse effects of lateral sway on the stability of individual columns as well as the structure as a whole, and the integrity of nonstructural partitions, glazing, and mechanical elements in the building. No systematic study has yet been published to determine the precise relationship between drift and the above factors. Cracking associated with lateral deflections of nonstructural elements such as partitions, windows, etc., may cause serious maintenance problems (loss of acoustical properties, leakage, etc.). Therefore, a drift limitation should be selected to minimize such cracking.

In the absence of code limitations in the past, buildings were designed for wind loads with arbitrary values of drift, ranging from about 1/300 to 1/600, depending on the judgment of the engineer. Deflections based on drift limitation of about 1/300 used several decades ago were computed assuming the wind forces to be resisted by the structural frame alone. In reality, as mentioned previously, the heavy masonry partitions and exterior cladding common to buildings of that period considerably increased the lateral with walls connected by link bars (as illustrated in Fig. 10-32) is advantageous for irregular configurations in multistory structures. Once the loading on each wall is determined, wall stresses can be calculated and thicknesses modified, if necessary.

4. Application. Shear wall buildings are used in apartment, hotel, and other residential buildings where walls are customarily spaced between 15 and 24 ft apart with floor slab thicknesses proportioned according to span. Spans up to 40 ft have been used with prestressed hollow-core concrete slabs.

The shear wall structure is used in buildings where permanent partitions and the lack of flexibility for future modifications can be tolerated. Its major advantages lie in the speed of construction, low reinforcing steel content and acoustical privacy.

In current North American practice, shear wall buildings are mainly cast-in-place, but trends to systems building are leading to an increase in the number of buildings being constructed using large panel, precast components for floors and/or walls.

Shear wall structures are well suited for construction in earthquake areas, and they have performed well during recent disasters.[10-24, 10-25] While costs vary from city to city, shear wall buildings usually become economical as soon as lateral forces affect the design and proportioning of flat plate or beam and column structures. Buildings of up to 70 stories have been built using shear walls. Feasibility studies for projects up to 200 stories utilizing shear walls have been made and found workable.

5. Coupled Shear Walls Supported on Exterior Columns Only. Parking areas under residential buildings require different spans from the apartments above. For this reason the shear walls must be stopped and supported on exterior columns, thus leaving the entire parking area column-free. The lower portions of such shear walls act as a deep beam spanning between the supporting columns.

The majority of shear wall buildings have coupled shear wall systems to accommodate corridors in the middle. A study carried out at the Portland Cement Association showed the feasibility of supporting a coupled shear wall on exterior columns as shown in Fig. 10-25. The computer study showed that the second floor beam (supporting the shear walls) acts like the tension member for the coupled shear wall above. The lintels over the doors for the next five stories act as compression struts; above that level the forces in the lintels are minimal, as can be seen in Fig. 10-25. The study also indicated that the shear wall must be supported during construction only until about the fifth floor is cast; after that the structure is self-supporting.

EXAMPLE 10-1: The 20-story apartment building designed with coupled shear walls is shown in its typical plan and elevation in Fig. 10-26. The typical wall section shows the pair of coupled shear walls

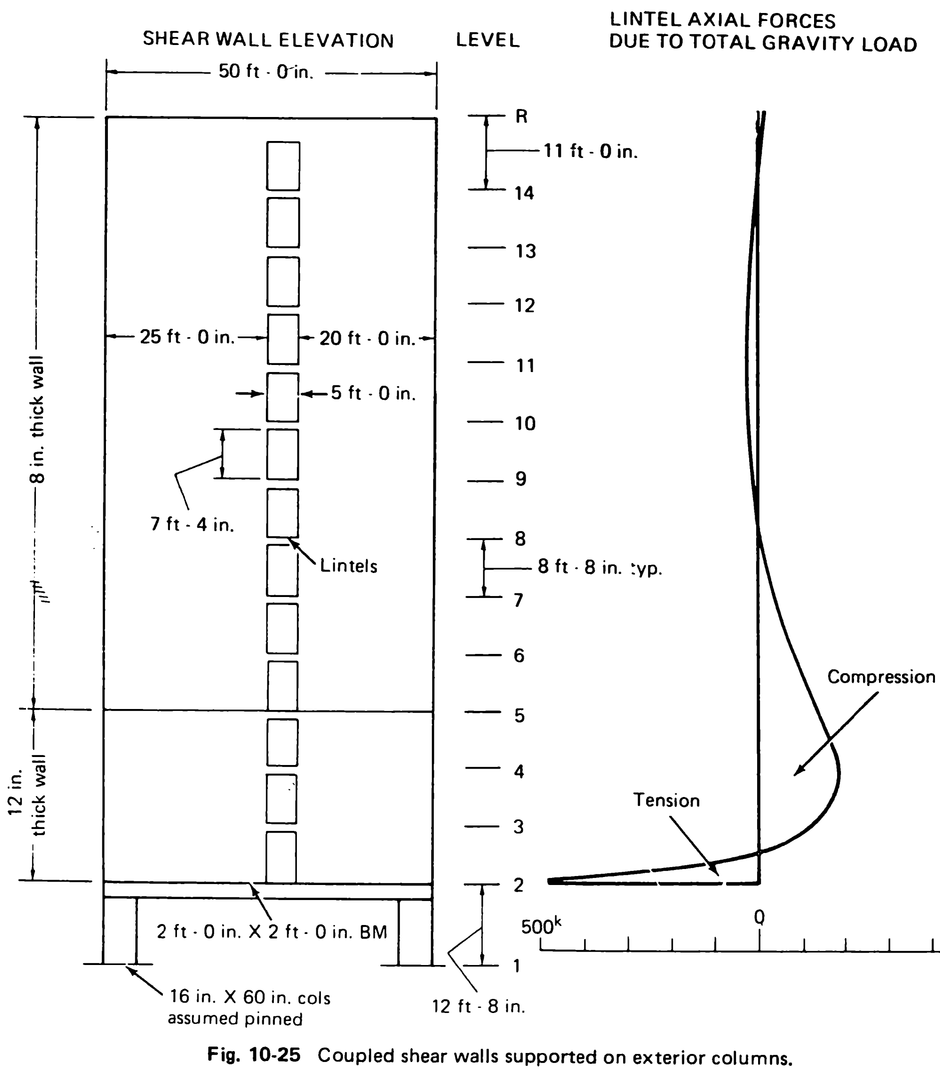

Fig. 10-25 Coupled shear walls supported on exterior columns.

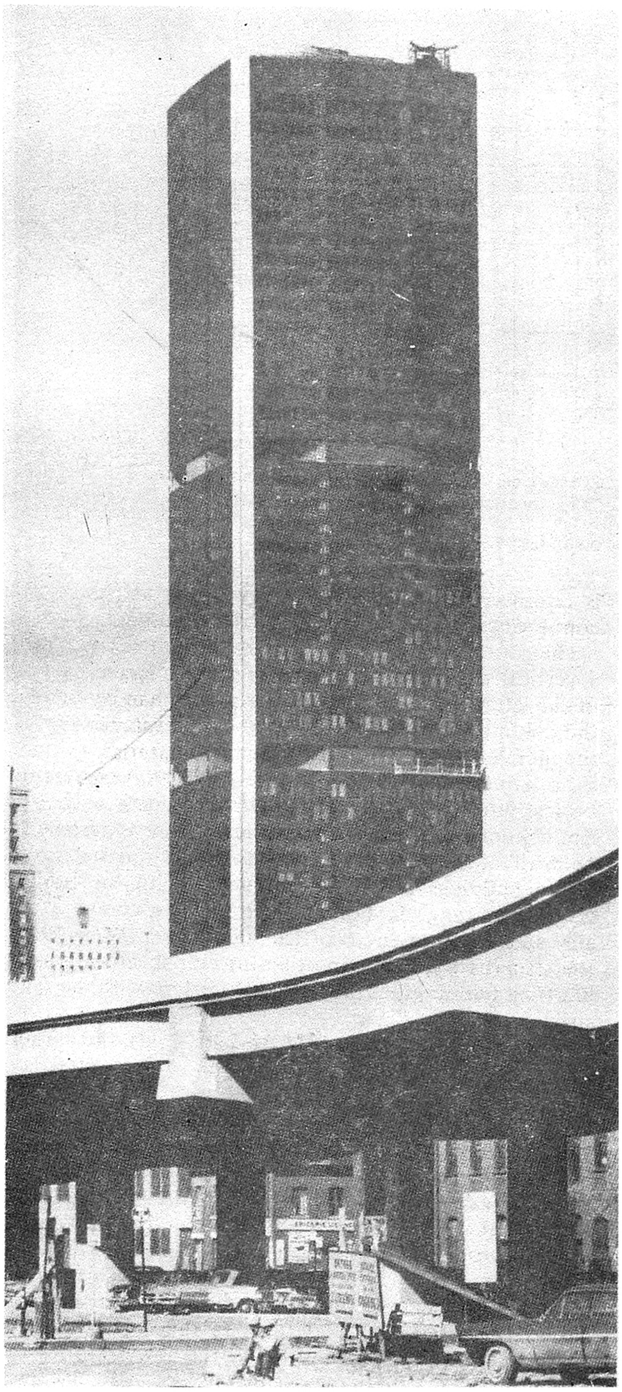

Fig. 10-34 The 47-story-high Place Victoria, Montreal, Canada.

the combination of gravity (dead) load and overturning wind moments. If there are resulting tensile forces, they must be resisted either by mobilizing load from adjacent columns using transfer girders (usually in the basement), or by anchoring the shear walls into the foundation medium. The economic consequences of the tensile forces should be investigated. If they are unavoidable, the size or number of shear walls needs to be increased.

Structures or elements of structures in which the ratio of wind stress to dead load stress is high are very sensitive to wind.

The most economical shear wall–frame structure (no premium for height) is achieved when there is a proper balance between the gravity load (dead load) and the overturning moment on each of the shear walls; ideally, if the overturning stresses can be accommodated within the 33% increase in the allowable gravity load stresses. Such a condition is achieved in shear wall structures where the shear walls carry the entire gravity load. At the other extreme, many buildings have only a central core as a shear wall that carries sometimes only a small portion of the dead load while resisting the majority of the overturning moments. If such buildings have a properly balanced shear wall-frame interaction, they may have sufficient rigidity to resist lateral forces. However, cases have been observed where a slip-formed core had around it a one-bay frame in which the flat plate had neither sufficient connection to the exterior columns nor a moment connection to the core; the result was an intolerable flexibility of the total building in response to wind.

EXAMPLE 10-2: Shear wall-frame interaction. The objective of this example is to determine, through a step-by-step optimization, the minimum amount of shear walls required for a given apartment building. The typical half-floor plan, section and column sizes (36-story building) are shown in Fig. 10-35. The basic structural system denoted as structure "A" consists of a centrally located corewall extending throughout the entire height of the building and 10 three-bay open frames in the transverse direction. The assumed uniform wind load is 20 psf on the 60 × 220 ft building.

The structure was analyzed for wind using the computer program described in Ref. 10-19. Although structure "A" has more than sufficient stiffness (the computed drift is 1/850), the net tension in the extreme windward "fiber" at the base of the corewall is 700 psi.

To reduce this tension in the corewall, a 20-ft-wide, 24-story-high shear wall was introduced in an exterior bay of one of the open frames as shown in Fig. 10-36(a). This new structural layout was denoted structure "B". The analysis of this case (see Table 10-3) shows that although the tension at the base of the corewall is reduced significantly–to 395 psi from 700 psi in "A"–substantial tensile stresses occur at the base of the added shear wall (515 psi). This indicates that the added shear wall, due to its stiffness, attracted a high moment relative to its dead load–resulting in significant net tensile stresses.

In an effort to further reduce the tension at the base of the corewall a third structure, structure "C," with a pair of 20-ft-wide, 18-story-high shear walls along an exterior column line, Fig. 10-36(b), was next analyzed. As might be expected, this further stiffened the structure, bringing the drift (deflection index) down from 1/873 for structure "B" to 1/928. In addition, the stress at the base of the corewall was reduced to the point where a net compressive stress of 30 psi occurs in the extreme windward fiber. However, the tensile stresses at the base of the additional shear walls have increased to 615 psi–from 515 psi in structure "B."

When the shear walls in structure "C" are assumed to be located along an interior column line, such as line 2 in Fig 10-36(b), the net tensile stress at the base is reduced from 615 psi to 192 psi (as shown for the case "C-1" in Table 10-3). This is due to the added dead load on the shear wall. In structure "C" and "C-1" only the slab strips were considered as linking the additional pair of shear walls

Structure "D," a fourth structure considered, is essentially the same as structure "C-1" except that beams were introduced to link the additional shear walls along column-line 2 such that the stiffness of the coupling elements connecting the pair of shear walls is three times that of the slab strips in "C." Table 10-3 indicates that the increase in stiffness of the coupling between the pair of shear walls not only increased the compressive stress at the base of the corewall, but also slightly decreased the tensile stress under the additional (coupled) shear walls. The decrease of tensile stress in the coupled shear walls is only slight, since the reduced shear wall moments are accompanied by axial forces (tension and compression) resulting from coupling.

It is obvious that the tensile force resulting from the net tensile stresses has to be either anchored into the foundation material, or it must be shifted with the help of shear beams to the neighboring columns to be overcome by their gravity loads. If the tensile load

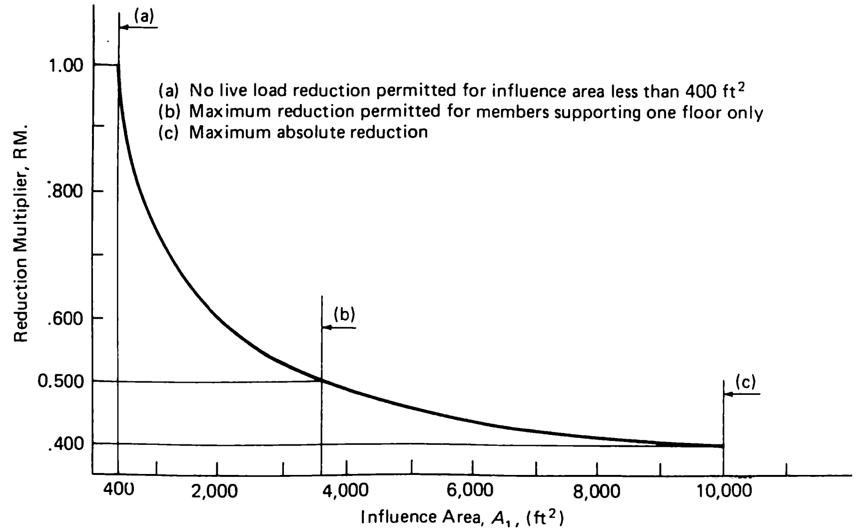

Fig. 10-46 Reduction multiplier (*RM*) for live loads $\left(0.25 + \frac{15}{\sqrt{A_1}}\right)$

under some circumstances. It is used in many and varied construction applications; however, for multistory construction its use may be particularly attractive. The use of normal weight versus lightweight concrete in multistory construction involves the study of several variables. In most areas of the country the lightweight concrete used in buildings has a weight of 110-115 lb per cubic yard, since the lightweight fines are replaced by natural sand.

The reduced dead load resulting from using lightweight concrete produces structural advantages such as: (a) a reduction of sizes of flexural members, columns, and foundations; (b) equivalent fire ratings obtained with thinner lightweight concrete sections; and (c) lower inertia forces in earthquake design.

The use of lightweight concrete for high-rise buildings was extremely popular in many cities of North America during the 1950s and 1960s. At that time the cost differential between normal and lightweight concrete was very low. In New York, for example, at times there was no cost premium for lightweight concrete, and in some metropolitan areas the cost differential was about $2.50 per cubic yard of concrete. As a result, many high-rise buildings were constructed with lightweight concrete.

With the drastic changes in energy costs following the oil embargo of 1973, prices for lightweight aggregates have substantially increased because most of them are manufactured by heat processes. As a consequence, the use of lightweight concrete in high-rise construction has declined.

Under the present circumstances, the weight reduction (when using lightweight concrete) must produce substantial savings in slab and column reinforcement to warrant the cost premium.

10.6.6 Transfer Girders

In recent years many buildings have been constructed with mixed occupancies, such as apartments or hotels in the upper stories while the lower stories contained commerical space, theaters, schools, or other nonresidential space. Additionally, in large metropolitan locations most of the new buildings contain parking garages in the lower stories.

TABLE 10-4 Live Load Reduction Multipliers (*RM*) for Columns of a 7-Story Building (25-ft square bays)

	Interior Columns		*Edge Columns*		*Corner Columns*	
Story	A_1	*RM*	A_1	*RM*	A_1	*RM*
7 (Roof)		*		*		*
6	2500	0.550	1250	0.674	625	0.850
5	5000	0.462	2500	0.550	1250	0.674
4	7500	0.423	3750	0.495	1875	0.596
3	10000	0.400**	5000	0.462	2500	0.550
2	12500	0.400**	6250	0.439	3125	0.518
1	15000	0.400**	7500	0.423	3750	0.495

**No reduction permitted for roof live loads.*
***Maximum reduction permitted.*