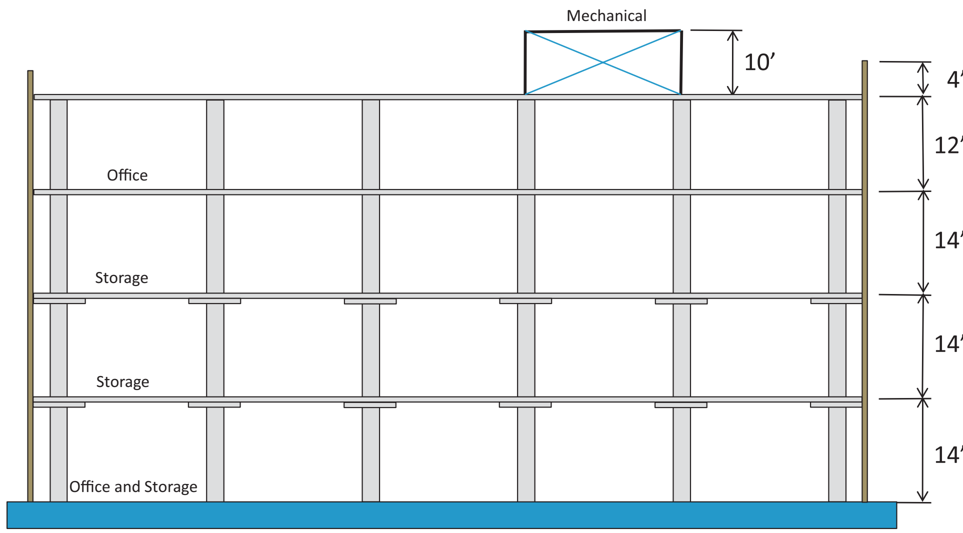
Fig. G16-1, cont'd



(c) Elevation

density of 90 lb/ft^3. These panels have window openings that cover approximately 35% of the façade. These window areas weigh 12 lb/ft^2. The exterior wall extends 4 ft above the roof to form a solid parapet. The panels are supported vertically at grade and at levels 2, 3, and 4. The detailing of the panel connections is such that the panels are considered as effective seismic weight in each direction.

The books are stored in plastic containers, which in turn are supported by a steel rack system. The racks cover approximately 70% of the floor area. Small forklifts (not to be classified as permanent equipment) are used to place and remove pallets of containers from the shelves. The design live load for the book storage area is 150 lb/ft^2. The rack storage system, which is anchored to the slab, weighs approximately 20 lb/ft^2. The racking system is laterally braced in two orthogonal directions with steel X-bracing. The system is sufficiently rigid to transfer the storage loads to the floor slabs.

The office area on the fourth floor of the building is designed for a live load of 50 lb/ft^2. Various work spaces are formed by a combination of fixed and movable partitions. A partition allowance of 15 lb/ft^2 is used in the design of the office floors. Two-thirds of this value, 10 lb/ft^2, is used for effective seismic weight as allowed by item 2 in Section 12.7.2.

The design dead load value used for the ceiling and mechanical areas of the main building is 15 lb/ft^2. Floor finishes in the office area are assumed to weigh 2.5 lb/ft^2. The floors in the storage areas are bare concrete.

The second and third floors have two openings, one (15 ft × 20 ft) to accommodate a hydraulic elevator for use in transporting the books (including the small forklifts), and the other (15 ft × 10 ft) for an elevator that services the offices on the fourth floor. The fourth floor and roof have only the smaller opening. Other minor openings exist in the floors and roof, but these openings are small and are not considered when computing the effective seismic weight. Two stairwells are also present in the building (not shown on plan), but the weights of these are, on a pound per square foot basis, approximately the same as the floor slab.

The mechanical room contains various heating, air conditioning, and ventilating equipment. The average dead load for the entire mechanical room, including the steel framing, roof, and equipment, is 60 lb/ft^2. The roofing over the remainder of the building (that area not covered by the mechanical room) is assumed to weigh 15 lb/ft^2.

The ground snow load for the building site is 60 lb/ft^2. Based on the procedures outlined in Chapter 7 of ASCE 7, the flat roof snow load is 42 lb/ft^2.

Calculation of the effective seismic weight, W, is based on the requirements of Section 12.7.2. The weight includes all dead load, a minimum of 25% of the floor live load in the storage areas, a 10 lb/ft^2 partition allowance where appropriate, total operating weight of permanent equipment, and 20% of the uniform design snow load when the flat roof snow load exceeds 20 lb/ft^2. Each of these load types is pertinent to the building under consideration.

Dead Load

The seismic load for the first floor level (a slab on grade) transfers directly into the foundation, so this load need not be considered as part of the effective seismic weight.

The loading for the second floor consists of the slab, drop panels, columns, storage rack system, ceiling and mechanical system, and exterior cladding.

Slab:	Total area = $160 \times 110 - 15 \times 20 - 15 \times 10 = 17{,}150$ ft^2, Unit weight = $(9/12) \times 115 = 86.2$ lb/ft^2, and Weight = $17{,}150 \times 86.2/1{,}000 = 1{,}478$ kip.
Drop panels:	30 panels × 100 ft^2 per panel = 3,000 ft^2, Unit weight = $(3/12) \times 115 = 28.8$ lb/ft^2, and Weight = $3{,}000 \times 28.8/1{,}000 = 86$ kip.
Columns:	Clear height tributary to first story = 13 ft, Clear height tributary to second story = 13 ft, Height tributary to second level = $(13+13)/2 = 13$ ft, Column area = 4.91 ft^2, and Weight = 30 columns × $4.91 \times 13 \times 115/1{,}000 =$ 220 kip.
Storage rack system:	Total area = 17,150 ft^2 (no deduction taken for columns), Effective area = $0.7 \times 17{,}150 = 12{,}005$ ft^2, Unit weight = 20 lb/ft^2, and Weight = $12{,}005 \times 20/1{,}000 = 240$ kips.
Ceiling and mechanical system:	Total area = 17,150 ft^2 (no deduction taken for columns), Unit weight = 15 lb/ft^2, and Weight = $17{,}150 \times 15/1{,}000 = 257$ kip.
Exterior cladding:	Perimeter = $2(160 + 110) = 540$ ft, Height tributary to second level = 14 ft,

Area of 4-in.-thick precast = $0.65 \times 540 \times 14$ = 4,914 ft^2,
Unit weight of panel = $(4/12) \times 90$ = 30.0 lb/ft^2,
Total panel weight = $4{,}914 \times 30.0/1{,}000$ = 147 kip,
Area of glass windows = $0.35 \times 540 \times 14$ = 2,646 ft^2,
Unit weight of glass = 12 lb/ft^2,
Total glass weight = $2{,}646 \times 12/1{,}000$ = 32 kip, and
Total cladding weight = 147 + 32 = 179 kip.

Total dead load at second level = 1,478 + 86 + 220 + 240 + 257 + 179 = 2,460 kip.

The dead load on the third level is almost identical to that on the second level. The only difference is that the absence of the drop panels at the fourth story has a slight influence on the clear length of the columns at the third story. For this example, this small difference is ignored, and the same dead load is used for the second and third levels.

The dead load for the fourth level is computed as follows:

Slab:	Total area = $160 \times 110 - 15 \times 10$ = 17,450 ft^2, Unit weight = $(9/12) \times 115$ = 86.2 lb/ft^2, and Weight = $17{,}450 \times 86.2/1{,}000$ = 1,504 kip.
Columns:	Clear height tributary to third story = 13.25 ft, Clear height tributary to fourth story = 11.25 ft, Height tributary to second level = (13.25 + 11.25)/2 = 12.25 ft, Column area = 4.91 ft^2, and Weight = 30 columns $\times 4.91 \times 12.25 \times 115/1{,}000$ = 208 kip.
Partitions:	Total area = 17,450 ft^2 (no deduction taken for columns), Unit weight = 10 lb/ft^2 (see Section 12.7.2, item 2), and Weight = $17{,}450 \times 10/1{,}000$ = 175 kip.
Floor finish:	Total area = 17,450 ft^2 (no deduction taken for columns), Unit weight = 2.5 lb/ft^2, and Weight = $17{,}450 \times 2.5/1{,}000$ = 44 kip.
Ceiling and mechanical system:	Total area = 17,450 ft^2 (no deduction taken for columns), Unit weight = 15 lb/ft^2, and Weight = $17{,}450 \times 15/1{,}000$ = 262 kip.
Cladding:	Perimeter = 2(160 + 110) = 540 ft, Height tributary to fourth level = 0.5(14 + 12) = 13 ft, Area of 4-in.-thick precast = $0.65 \times 540 \times 13$ = 4,563 ft^2,

Unit weight of panel = (4/12) × 90 = 30 lb/ft²,
Total panel weight = 4,563 × 30/1,000 = 137 kip,
Area of glass windows = 0.35 × 540 × 13 = 2,457 ft²,
Unit weight of glass = 12 lb/ft²,
Total glass weight = 2,457 × 12/1,000 = 29 kip, and
Total cladding weight = 137 + 29 = 166 kip.

Total dead load at fourth level = 1,504 + 208 + 175 + 44 + 262 + 166 = 2,359 kip.

The dead load for the roof level is computed as follows:

Slab:	Total area = 160 × 110 – 15 × 10 = 17,450 ft², Unit weight = (9/12) × 115 = 86.2 ft², and Weight = 17,450 × 86.2/1,000 = 1,504 kip.
Columns:	Clear height at fourth story = 11.25 ft, Height tributary to second level = (11.25)/2 = 5.62 ft, Column area = 4.91 ft², and Weight = 30 columns × 4.91 × 5.62 × 115/1,000 = 95 kip.
Ceiling and mechanical system:	Total area = 17,450 ft² (no deduction taken for columns), Unit weight = 15 lb/ft², and Weight = 17,450 × 15/1,000 = 262 kip.
Roofing:	Total area of main roof = 160 × 110 – 40 × 35 = 16,200 ft², Unit weight = 15 lb/ft², and Weight = 16,200 × 15/1,000 = 243 kip.
Mechanical area:	Total area = 40 × 35 = 1,400 ft², Unit weight = 60 lb/ft² (estimated), and Weight = 1,400 × 60/1,000 = 84 kip.
Cladding:	Perimeter = 2(160 + 110) = 540 ft, Height tributary to roof = 6 ft, Area of 4-in.-thick precast = 0.65 × 540 × 6 = 2,106 ft², Unit weight of precast = (4/12) × 90 = 30 lb/ft², Total panel weight = 2,106 × 30/1,000 = 63 kip, Area of glass windows = 0.35 × 540 × 6 = 1,134 ft², Unit weight of glass = 12 lb/ft², Total glass weight = 1,134 × 12/1,000 = 14 kip, and Total cladding weight = 63 + 14 = 77 kip.
Parapet:	Perimeter = 2(160 + 110) = 540 ft, Height tributary to roof = 4 ft, Area of 4-in.-thick precast = 540 × 4 = 2,160 ft²,

Unit weight of precast = (4/12) × 90 = 30 lb/ft^2, and

Total parapet weight = 2,160 × 30/1,000 = 65 kip.

Total dead load at the roof = 1,504 + 95 + 262 + 243 + 84 + 77 + 65 = 2,330 kip.

The total dead weight for the building, including the mechanical level, is 9,609 kip. Using a building volume exclusive of the mechanical area of (160 × 110) × 54 = 950,400 ft^3, the dead load density for the building is 9,609/950,400 = 0.0101 kip/ft^3 or 10.0 lb/ft^3. This weight is a bit heavier than that which would be appropriate for a low-rise office building, but it is reasonable for a concrete warehouse building. Calculation of building density is a good reality check on effective seismic weight. Low-rise buildings generally have a density in the range of 7 to 10 lb/ft^3, depending on material and use.

Contribution from Storage Live Loads at Levels 2 and 3

As mentioned in the building description, the building has a design storage live load of 150 lb/ft^2. However, only 70% of each floor is reserved for storage, and the remainder is used for aisles, stairs, and restrooms. The openings for elevators are considered separately.

Building use statistics indicate that the storage racks are near capacity in the summer months when school is not in session and reduce to about 30% capacity during the fall and winter months. Section 12.7.2 states that a minimum of 25% of storage live load shall be used as effective seismic weight. For this facility, the 25% minimum is used. However, others might argue that, on the basis of use statistics, a larger portion of the load should be used.[1]

The live load contribution to effective seismic weight is as follows for the second and third levels:

Total area = 160 × 110 − 15 × 20 − 15 × 10 = 17,150 ft^2,
Effective area = 0.7 × 17,150 = 12,005 ft^2,
Effective live load = 0.25(150) = 37.5 lb/ft^2, and
Total live load contribution to seismic weight = 12,005 × 37.5/1,000 = 450 kip.

Contribution of Snow Load at Roof Level

Section 12.7.2 indicates that 20% of the uniform design snow load must be included in the effective seismic weight when the flat roof snow load exceeds 30 lb/ft^2. The flat roof snow load for this building is 42 lb/ft^2, so snow load

[1] The design of combined building-rack storage systems is considerably more complex than indicated in this example. See Chapters 13 and 15 of ASCE 7 for requirements for the design and attachment of rack systems to the building superstructure. See also the *Specifications for the Design, Testing, and Utilization of Industrial Steel Storage Racks* (RMI 2009).

Table G16-1 Summary of Effective Seismic Weight Calculations

	Load Contribution (kip)			
Level	Dead	Live	Snow	Total
2	2,460	450	0	2,910
3	2,460	450	0	2,910
4	2,359	0	0	2,359
R	2,330	0	148	2,478
Total	9,609	900	148	10,657

must be included. The building has a flat roof (both the main roof and the mechanical room), so the uniform snow load is 42 lb/ft^2. Using a total area of $160 \times 110 = 17{,}600$ ft^2, the contribution from snow to the effective seismic weight is

Total area = 17,600 ft^2,
Effective snow load = 0.2(42) = 8.4 lb/ft^2, and
Total snow load contribution to seismic weight = $17{,}600 \times 8.4/1{,}000$
= 148 kip.

Table G16-1 summarizes the effective seismic weight for the entire system. The design seismic base shear [Eq. (12.8-1)] should be based on these weights, as should the distribution of forces along the height of the building [Eqs. (12.8-11) and (12.8-12)]. These forces should be placed at the center of mass of floors of the building, as appropriate. For this building, the center of mass is slightly offset from the plan center because of the floor openings and the somewhat eccentric location of the mechanical room.

The weights shown in **Table G16-1** are to be used in an ELF analysis of the system. If a three-dimensional rigid-diaphram modal analysis is used, the mass moments of inertia are required for each floor. When heavy cladding is used, including this cladding as line masses situated at the perimeter may be appropriate.

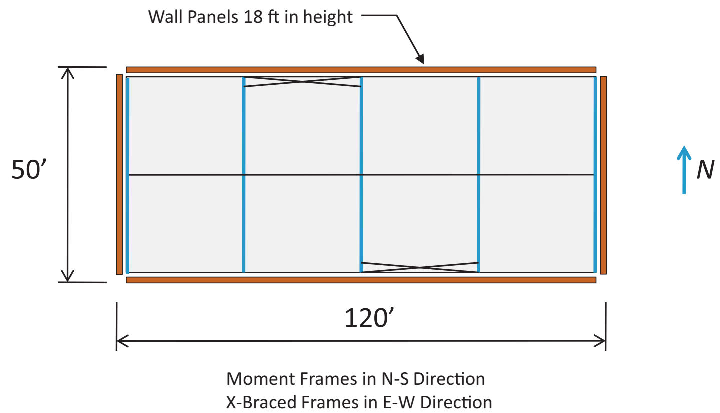
If the mechanical penthouse covered more area, considering this as a separate level of the building may be appropriate. Section 12.2.3.1 covers situations when different lateral load–resisting systems are used along the height of the building. The light rooftop structure used in this Seismic Design Category B example is exempt from the requirements of 12.2.3.1.

Consideration should also be given to the design, detailing, and anchorage of the steel rack system used in this building. Chapters 13 and 15 provide the requirements for the analysis and design of the system.

Low-Rise Industrial Building

In the previous example, the weight of the cladding parallel and perpendicular to the direction of loading was included in the effective seismic weight in each direction. Thus, the effective seismic weight is the same in each direction.

Fig. G16-2
Plan view of a one-story industrial building



For low-rise buildings, typically one story, the cladding panels may be detailed such that they are self-supporting when resisting seismic loads parallel to the plane of the panels and hence do not contribute to the seismic resistance of the main structural system when seismic loads act parallel to the plane of the panels. However, the effective weight of panels perpendicular to the direction of loading must be included.

Consider the low-rise industrial building shown in **Fig. G16-2**. The siding for the building consists of 5-in.-thick insulating concrete sandwich panels that weigh 60 lb/ft^2. On the west face, openings cover approximately 35% of the wall panel area. The other faces have only minor window and door openings, and these openings are ignored in computing the effective seismic weight. Only the panel weight is considered herein.

For seismic forces in the north–south direction, the panel contribution to the effective seismic weight is based on one-half of the weight of the two 120-ft-long walls:

$$W_{\text{panels,N-S}} = 2 \times 120 \text{ ft} \times 18 \text{ ft} \times 60 \text{ lb/ft}^2 \times 0.5 = 129{,}600 \text{ lb} = 130 \text{ kip}.$$

For seismic loads in the east–west direction, the effective seismic weight is

$$W_{\text{panels,E-W}} = (1 + 0.65) \times 50 \text{ ft} \times 18 \text{ ft} \times 60 \text{ lb/ft}^2 \times 0.5 = 44{,}550 \text{ lb} = 45 \text{ kip}.$$

The 0.5 in the aforementioned calculations is based on half of the total effective seismic panel weight being carried by the steel roof framing, and the remainder being resisted at the foundation.

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Example 17

Period of Vibration

This example explores computing the period of vibration of building structures. This example reviews the empirical methods that ASCE 7 provides for computing periods, computes periods for a few simple buildings, and provides a more detailed analysis wherein the empirical periods are compared with the period based on rational analysis.

Approximate Fundamental Period T_a

Section 12.8.2.1 addresses computing the approximate period of vibration of buildings. Three basic formulas are provided:

$$T_a = C_t h_n^x \qquad \text{(Eq. 12.8-7)}$$

$$T_a = 0.1\,N \qquad \text{(Eq. 12.8-8)}$$

$$T_a = \frac{0.0019}{\sqrt{C_w}} h_n \qquad \text{(Eq. 12.8-9)}$$

These formulas are highly empirical and are to be used for seismic analysis of building structures only. Eq. (12.8-7) applies to all buildings, Eq. (12.8-8) applies to certain moment frames, and Eq. (12.8-9) applies only for masonry or concrete shear wall structures. The primary use of T_a is in computing seismic base shear V. The period (in relation to T_S) is also used to determine the appropriate method of analysis (Table 12.6-1).

In Eq. (12.8-7), the coefficient C_t and the exponent x come from Table 12.8-2 and depend on the structural system and structural material. These terms were developed from regression analysis of the measured periods of real buildings in California. The coefficients for buckling-restrained brace (BRB) systems are the same as those for eccentrically braced frames (C_t = 0.03, x = 0.75). These coefficients produce a somewhat longer period than those obtained for braced frames (all other systems in the table). This is logical because the cross-sectional area of the core bracing elements in BRB systems is always smaller than that required for traditional braced frames. Note also that the coefficients used for eccentrically braced steel frames apply only if the eccentrically braced frame is designed and detailed according to the requirements of the *Seismic Provisions for Structural Steel Buildings* (AISC 2010b).

When applying Eq. (12.8-7), the basic uncertainty is in the appropriate value to use for the structural height h_n, which is defined as "the vertical distance from the base to the highest level of the seismic force resisting system of the structure." Section 11.2 of ASCE 7 defines the base as "the level at which the horizontal seismic ground motions are considered to be imparted to the structure." For a building on level ground without basements, the base may be taken as the grade level. In many cases, however, establishing the exact location of the base may not be easy. This is particularly true when the building is constructed on a sloped site, or when one or more basement levels exist. Also of some concern is the definition of the highest level of the structure. This height should not include small mechanical rooms or other minor rooftop appurtenances. For buildings with sloped roofs, the structural height should be taken from the base to the average height of the roof. See also Commentary Section 12.2 for definitions and illustrations of the base.

Consider, for example, the X-braced steel frame structures shown in **Fig. G17-1**. Structure (a) has no basement. Here, the height h_n is the distance from the grade level to the roof, not including the penthouse. Structure (b) is like structure (a), but has a full basement. At the grade level, the slab is thickened, and horizontal seismic force at the grade level is partially transferred though the diaphragms to exterior basement walls. Here again, the effective height should be taken as the distance from the grade level to the main roof. However, a computer model would produce a longer period for structure (b) compared with structure (a) because of the axial deformations that occur in the subgrade braces and columns of the braced frame of structure (b). In structure (c), the first grade slab is not thickened, and the braced frame shear forces are not expected to completely transfer out through the first-floor diaphragm. However, the soils adjacent to the basement walls

Fig. G17-1
Finding the effective height h_n for a braced frame