12

NONBUILDING STRUCTURE DESIGN

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Chapter 14 of the 2000 *NEHRP Recommended Provisions* and *Commentary* (hereafter, the *Provisions* and *Commentary*) is devoted to nonbuilding structures. Nonbuilding structures comprise a myriad of structures constructed of all types of materials with markedly different dynamic characteristics and a wide range of performance requirements.

Nonbuilding structures are a general category of structure distinct from buildings. Key features that differentiate nonbuilding structures from buildings include human occupancy, function, dynamic response, and risk to society. Human occupancy, which is incidental to most nonbuilding structures, is the primary purpose of most buildings. The primary purpose and function of nonbuilding structures can be incidental to society or the purpose and function can be critical for society.

In the past, many nonbuilding structures were designed for seismic resistance using building code provisions developed specifically for buildings. These code provisions were not adequate to address the performance requirements and expectations that are unique to nonbuilding structures. For example consider secondary containment for a vertical vessel containing hazardous materials. Nonlinear performance and collapse prevention, which are performance expectations for buildings, are inappropriate for a secondary containment structure, which must not leak.

Traditionally, the seismic design of nonbuilding structures depended on the various trade organizations and standards development organizations that were disconnected from the building codes. The *Provisions* have always been based upon strength design and multiple maps for seismic ground motion definition, whereas most of the industry standards were based on allowable stress design and a single zone map. The advent of the 1997 *Provisions* exacerbated the problems of the disconnect for nonbuilding structures with direct use of seismic spectral ordinates, and with the change to a longer recurrence interval for the seismic ground motion. It became clear that a more coordinated effort was required to develop appropriate seismic design provisions for nonbuilding structures.

This chapter develops examples specifically to help clarify Chapter 14 of the *Provisions*. The solutions developed are not intended to be comprehensive but instead focus on interpretation of *Provisions* Chapter 14 (Nonbuilding Structure Design Requirements). Complete solutions to the examples cited are beyond the scope of this chapter.

Although this volume of design examples is based on the 2000 *Provisions*, it has been annotated to reflect changes made to the 2003 *Provisions*. Annotations within brackets, [ ], indicate both organizational changes (as a result of a reformat of all of the chapters of the 2003 *Provisions*) and substantive technical changes to the 2003 *Provisions* and its primary reference documents. While the general concepts of the changes are described, the design examples and calculations have not been revised to reflect the changes to the 2003 *Provisions*.

Several noteworthy changes were made to the nonbuilding structures requirements of the 2003 *Provisions*. These include clearer definition of the scopes of Chapters 6 and 14, expanded, direct definition of structural systems (along with design parameters and detailing requirements) in Chapter 14, and a few specific changes for particular nonbuilding structural systems.

In addition to changes *Provisions* Chapter 14, the basic earthquake hazard maps were updated, the redundancy factor calculation was completely revised, and the minimum base shear equation for areas without near-source effects was eliminated.

Where they affect the design examples in this chapter, significant changes to the 2003 *Provisions* and primary reference documents are noted. However, some minor changes to the 2003 *Provisions* and the reference documents may not be noted.

In addition to the *Provisions* and *Commentary*, the following publications are referenced in this chapter:

United States Geological Survey, 1996. *Seismic Design Parameters* (CD-ROM) USGS.

> [The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

American Water Works Association. 1996. Welded Steel Tanks for Water Storage. AWWA.

American Petroleum Institute (API), Welded steel tanks for oil storage. API 650, 10th Edition, November 1998.

12.1 NONBUILDING STRUCTURES VERSUS NONSTRUCTURAL COMPONENTS

Many industrial structures are classified as either nonbuilding structures or nonstructural components. This distinction is necessary to determine how the practicing engineer designs the structure. The intent of the *Provisions* is to provide a clear and consistent design methodology for engineers to follow regardless of whether the structure is a nonbuilding structure or a nonstructural component. Central to the methodology is how to determine which classification is appropriate.

The design methodology contained in *Provisions* Chapter 6, Architectural, Mechanical, and Electrical Components Design Requirements, focuses on nonstructural component design. As such, the amplification by the supporting structure of the earthquake-induced accelerations is critical to the design of the component and its supports and attachments. The design methodology contained in *Provisions* Chapter 14 focuses on the direct effects of earthquake ground motion on the nonbuilding structure.

Table 12-1 Applicability of the Chapters of the *Provisions*

Supporting Structure	Supported Item	
	Nonstructural Component	Nonbuilding Structure
Building	Chapter 5 [4 and 5]for supporting structure Chapter 6 for supported item	Chapter 5 [4 and 5]for supporting structure Chapter 14 for supported item
Nonbuilding	Chapter 14 for supporting structure Chapter 6 for supported item	Chapter 14 for both supporting structure and supported item

The example shown in Figure 12-1 is a combustion turbine, electric-power-generating facility with four bays. Each bay contains a combustion turbine and supports an inlet filter on the roof. The uniform seismic dead load of the supporting roof structure is 30 psf. Each filter weighs 34 kips.

The following two examples illustrate the difference between nonbuilding structures that are treated as nonstructural components, using *Provisions* Chapter 6, and those which are designed in accordance with *Provisions* Chapter 14. There is a subtle difference between the two chapters:

> 6.1: ". . .if the combined weight of the supported *components* and *nonbuilding structures* with flexible dynamic characteristics exceeds 25 percent of the weight of the *structure*, the *structure* shall be designed considering interaction effects between the *structure* and the supported items."
>
> 14.4: "If the weight of a *nonbuilding structure* is 25 percent or more of the combined weight of the *nonbuilding structure* and the supporting *structure*, the design seismic forces of the *nonbuilding structure* shall be determined based on the combined *nonbuilding structure* and supporting structural system.. . . "

The difference is the plural *components* and the singular *nonbuilding structure*, and that difference is explored in this example.

[The text has been cleaned up considerably in the 2003 edition but some inconsistencies persist. Sec. 14.1.5 indicates the scopes of Chapters 6 and 14. Both chapters consider the weight of an individual supported component or nonbuilding structure in comparison to the total seismic weight. Where the weight of such an individual item does not exceed 25 percent of the seismic weight, forces are determined in accordance with Chapter 6. Where a nonbuilding structure's weight exceeds 25 percent of the seismic weight, Sec. 14.1.5 requires a combined system analysis and the rigidity or flexibility of the supported nonbuilding structure is used in determining the *R* factor. In contrast, Sec. 6.1.1 requires consideration of interaction effects only where the weight exceeds 25 percent of the seismic weight **and** the supported item has flexible dynamic characteristics.]

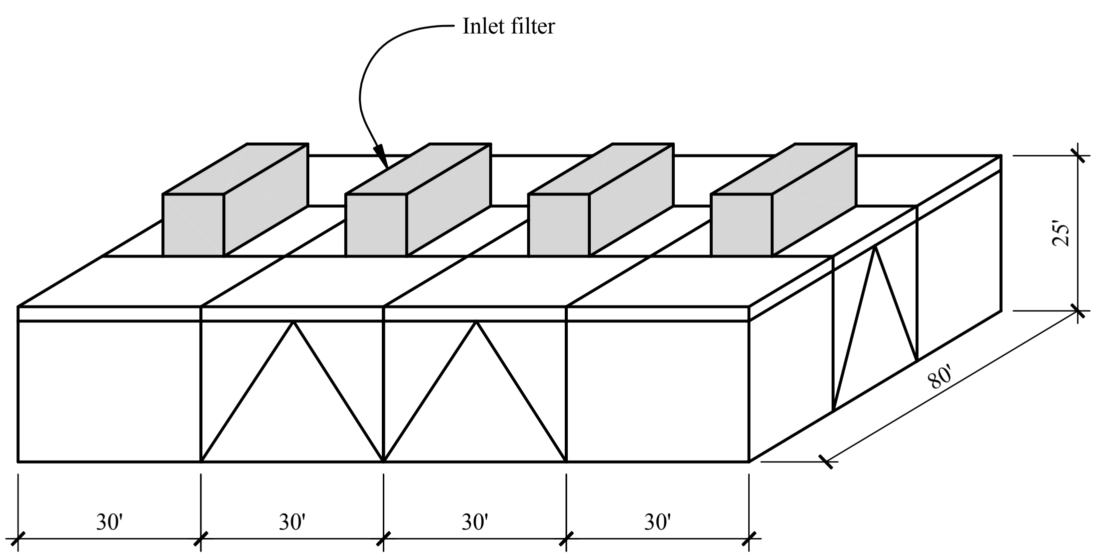


Figure 12-1 Combustion turbine building (1.0 ft = 0.3048 m).

12.1.1 Nonbuilding Structure

For the purpose of illustration assume that the four filter units are connected in a fashion that couples their dynamic response. Therefore, the plural components used in *Provisions* Sec. 6.1 is apparently the most meaningful provision.

[The text no longer contains a plural, but conceptually the frame could be considered a single item in this instance (just as the separate items within a single roof-top unit would be lumped together).]

12.1.1.1 Calculation of Seismic Weights

All four inlet filters = W_{IF} = 4(34 kips) = 136 kips

Support structure = W_{SS} = 4 (30 ft)(80 ft)(30 psf) = 288 kips

The combined weight of the nonbuilding structure (inlet filters) and the supporting structural system is

$$W_{combined} = 136 \text{ kips} + 288 \text{ kips} = 424 \text{ kips}$$

12.1.1.2 Selection of Design Method

The ratio of the supported weight to the total weight is:

$$\frac{W_{IF}}{W_{Combined}} = \frac{136}{424} = 0.321 > 25\%$$

Because the weight of the inlet filters is 25 percent or more of the combined weight of the nonbuilding structure and the supporting structure (*Provisions* Sec. 14.4 [14.1.5]), the inlet filters are classified as "nonbuilding structures" and the seismic design forces must be determined from analysis of the combined seismic-resistant structural systems. This would require modeling the filters, the structural components of the filters, and the structural components of the combustion turbine supporting structure to determine accurately the seismic forces on the structural elements as opposed to modeling the filters as lumped masses. [See the discussion added to Sec. 12.1.]

12.1.2 NONSTRUCTURAL COMPONENT

For the purpose of illustration assume that the inlet filters are independent structures, although each is supported on the same basic structure. In this instance, one filter is the nonbuilding structure. The question is whether it is heavy enough to significantly change the response of the combined system.

12.1.2.1 Calculation of Seismic Weights

One inlet filter = W_{IF} = 34 kips

Support structure = W_{SS} = 4 (30 ft)(80 ft)(30 psf) = 288 kips

The combined weight of the nonbuilding structures (all four inlet filters) and the supporting structural system is

$$W_{combined} = 4\ (34 \text{ kips}) + 288 \text{ kips} = 424 \text{ kips}$$

12.1.2.2 Selection of Design Method

The ratio of the supported weight to the total weight is:

$$\frac{W_{IF}}{W_{Combined}} = \frac{34}{424} = 0.08 < 25\%$$

Because the weight of an inlet filter is less than 25 percent of the combined weight of the nonbuilding structures and the supporting structure (*Provisions* Sec. 14.4 [14.1.5]), the inlet filters are classified as “nonstructural components” and the seismic design forces must be determined in accordance with *Provisions* Chapter 6. In this example, the filters could be modeled as lumped masses. The filters and the filter supports could then be designed as nonstructural components.

12.2 PIPE RACK, OXFORD, MISSISSIPPI

This example illustrates the calculation of design base shears and maximum inelastic displacements for a pipe rack using the equivalent lateral force (ELF) procedure.

12.2.1 Description

A two-tier, 12-bay pipe rack in a petrochemical facility has concentrically braced frames in the longitudinal direction and ordinary moment frames in the transverse direction. The pipe rack supports four runs of 12-in.-diameter pipe carrying naphtha on the top tier and four runs of 8-in.-diameter pipe carrying water for fire suppression on the bottom tier. The minimum seismic dead load for piping is 35 psf on each tier to allow for future piping loads. The seismic dead load for the steel support structure is 10 psf on each tier.

Pipe supports connect the pipe to the structural steel frame and are designed to support the gravity load and resist the seismic and wind forces perpendicular to the pipe. The typical pipe support allows the pipe to move in the longitudinal direction of the pipe to avoid restraining thermal movement. The pipe support near the center of the run is designed to resist longitudinal and transverse pipe movement as well as provide gravity support; such supports are generally referred to as fixed supports.

Pipes themselves must be designed to resist gravity, wind, seismic, and thermally induced forces, spanning from support to support.

If the pipe run is continuous for hundreds of feet, thermal/seismic loops are provided to avoid a cumulative thermal growth effect. The longitudinal runs of pipe are broken up into sections by providing thermal/seismic loops at spaced intervals. In Figure 12-2, it is assumed thermal/seismic loops are provided at each end of the pipe run.

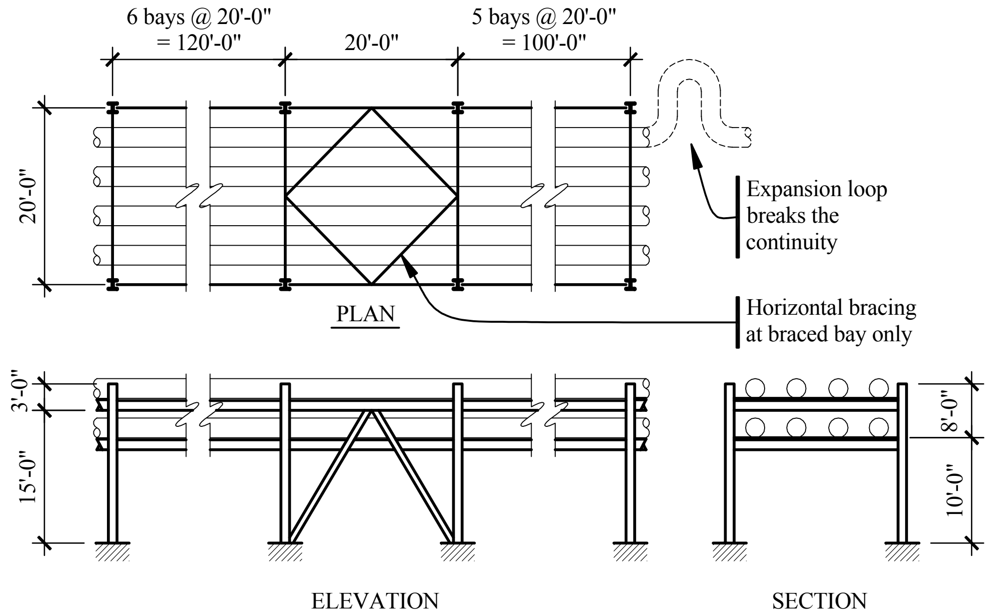


Figure 12-2 Pipe rack (1.0 ft = 0.3048 m).

12.2.2 *Provisions* Parameters

12.2.2.1 Ground Motion

The spectral response acceleration coefficients at the site are

$S_{DS} = 0.40$
$S_{D1} = 0.18$.

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

12.2.2.2 Seismic Use Group and Importance Factor

The upper piping carries a hazardous material (naphtha) and the lower piping is required for fire suppression. The naphtha piping is included in *Provisions* Sec. 1.3.1, Item 11 [Sec. 1.2.1, Item 11], therefore, the pipe rack is assigned to Seismic Use Group III.

According to *Provisions* Sec. 14.5.1.2 [14.2.1], the importance factor, I, is 1.5 based on Seismic Use Group, Hazard, and Function. If these three measures yield different importance factors, the largest factor applies.

12.2.2.3 Seismic Design Category

For this structure assigned to Seismic Use Group III with S_{DS} = 0.40 and S_{D1} = 0.18, the Seismic Design Category is D according to *Provisions* Sec. 4.2.1 [1.4].

12.2.3 Design in the Transverse Direction

[Chapter 14 has been revised so that it no longer refers to Table 4.3-1. Instead values for design coefficients and detailing requirements are provided with the chapter.]

12.2.3.1 Design Coefficients

Using *Provisions* Table 14.5.1.1 [14.4-2] (which refers to *Provisions* Table 5.2.2 [4.3-1]), the parameters for this ordinary steel moment frame are

$R = 4$
$\Omega_0 = 3$
$C_d = 3\frac{1}{2}$

[In the 2003 *Provisions*, R factor options are presented that correspond to required levels of detailing. R = 3.5, $\Omega = 3$; $C_d = 3$.]

Ordinary steel moment frames are retained for use in nonbuilding structures such as pipe racks because they allow greater flexibility for accommodating process piping and are easier to design and construct than special steel moment frames.

12.2.3.2 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.4}{4/1.5} = 0.15$$

From analysis, $T = 0.42$ sec. For nonbuilding structures, the fundamental period is generally approximated for the first iteration and must be verified with final calculations. For many nonbuilding structures the maximum period limit contained in the first paragraph of *Provisions* Sec. 5.4.2 [5.2.2] is not appropriate. As a result, the examples in this chapter neglect that limit. Future editions of the *Provisions* will clarify that this limit does not apply to nonbuilding structures. [In the 2003 *Provisions*, Sec. 14.2.9 makes clear that the approximate period equations do not apply to nonbuilding structures.]

Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.18}{0.42(4/1.5)} = 0.161$$

Using *Provisions* Eq. 5.4.1.1-3, C_s shall not be less than

$$C_s = 0.044IS_{DS} = 0.044(1.5)(0.4) = 0.0264$$

[This minimum C_s value has been removed in the 2003 *Provisions*. In its place is a minimum C_s value for long-period structures, which is not applicable to this example.]

Provisions Eq. 5.4.1.1-1 [5.2-2] controls; $C_s = 0.15$.

12.2.3.3 Seismic Weight

W = 2(20 ft)(20 ft)(35 psf + 10 psf) = 36 kips

12.2.3.4 Base Shear (*Provisions* Sec. 5.3.2 [5.2.1])

$V = C_s W$ = 0.15(36 kips) = 5.4 kips

12.2.3.5 Drift

Although not shown here, drift of the pipe rack in the transverse direction was calculated by elastic analysis using the design forces calculated above. The calculated lateral drift, δ_{xe} = 0.328 in.
Using *Provisions* Eq. 14.3.2.1 [5.2-15],

$$\delta_x = \frac{C_d \delta_{xe}}{I} = \frac{3.5(0.328 \text{ in.})}{1.5} = 0.765 \text{ in.}$$

The lateral drift must be checked with regard to acceptable limits. The acceptable limits for nonbuilding structures are not found in codes. Rather, the limits are what is acceptable for the performance of the piping. In general, piping can safely accommodate the amount of lateral drift calculated in this example. P-delta effects must also be considered and checked as required in *Provisions* Sec. 5.4.6.2 [5.2.6.2].

12.2.3.6 Redundancy Factor

Some nonbuilding structures are designed with parameters from *Provisions* Table 5.2.2 [4.3-1]; if they are termed "nonbuilding structures similar to buildings". For such structures the redundancy factor applies, if the structure is in Seismic Design Category D, E, or F. Pipe racks, being fairly simple moment frames or braced frames, are in the category similar to buildings. Because this structure is assigned to Seismic Design Category D, *Provisions* Sec. 5.2.4.2 [4.3.3.2] applies. The redundancy factor is calculated as

$$\rho = 2 - \frac{20}{r_{max_x}\sqrt{A_x}}$$

where $r_{\max_x}$ is the fraction of the seismic force at a given level resisted by one component of the vertical seismic-force-resisting system at that level, and A_x is defined as the area of the diaphragm immediately above the story in question. Some interpretation is necessary for the pipe rack. Considering the transverse direction, the seismic-force-resisting system is an ordinary moment resisting frame with only two columns in a single frame. The frames repeat in an identical pattern. The "diaphragm" is the pipes themselves, which are not rigid enough to make one consider the 240 ft length between expansion joints as a diaphragm. Therefore, for the computation of ρ in the transverse direction, each 20-by-20 ft bay will be considered independently.

The maximum of the sum of the shears in the two columns equals the story shear, so the ratio r_{max} is 1.0. The diaphragm area is simply the bay area:

A_x = 20 ft × 20 ft = 400 ft^{2,}

therefore,

$$\rho = 2 - \frac{20}{1.0\sqrt{400}} = 1.0$$

[The redundancy requirements have been changed substantially in the 2003 *Provisions*.]

12.2.3.7 Determining *E*

E is defined to include the effects of horizontal and vertical ground motions as follows:

$$E = \rho Q_E \pm 0.2\, S_{DS}\, D$$

where Q_E is the effect of the horizontal earthquake ground motions, which is determined primarily by the base shear just computed, and *D* is the effect of dead load. By putting a simple multiplier on the effect of dead load, the last term is an approximation of the effect of vertical ground motion. For the moment frame, the joint moment is influenced by both terms. *E* with the "+" on the second term when combined with dead and live loads will generally produce the largest negative moment at the joints, while *E* with the "-"on the second term when combined with the minimum dead load (0.9*D*) will produce the largest positive joint moments.

The *Provisions* also requires the consideration of an overstrength factor, Ω_0, on the effect of horizontal motions in defining *E* for components susceptible to brittle failure.

$$E = \rho\, \Omega_0\, Q_E \pm 0.2\, S_{DS}$$

The pipe rack does not appear to have components that require such consideration.

12.2.4 Design in the Longitudinal Direction

[In the 2003 *Provisions*, Chapter 14 no longer refers to Table 4.3-1. Instead, Tables 14.2-2 and 14.2-3 have design coefficient values and corresponding detailing requirements for each system.]

12.2.4.1 Design Coefficients

Using *Provisions* Table 14.5.1.1 [14.2-2] (which refers to *Provisions* Table 5.2.2 [4.3-1]), the parameters for this ordinary steel concentrically braced frame are:

$R = 4$
$\Omega_0 = 2$
$C_d = 4\frac{1}{2}$

[The 2003 *Provisions* allow selection of appropriate design coefficients and corresponding detailing for several systems. In the case of this example, *R* would equal 5, but the calculations that follow are not updated.]

Where *Provisions* Table 5.2.2 [4.3-1] is used to determine the values for design coefficients, the detailing reference sections noted in the table also apply. A concentric braced frame has an assigned *R* of 5, but an *R* of 4 is used to comply with *Provisions* Sec. 5.2.2.2.1 [4.3.1.2.1].

[In the 2003 *Provisions*, Chapter 14 no longer refers to Table 4.3-1. Instead, Tables 14.2-2 and 14.2-3 have design coefficient values and corresponding detailing requirements for each system. Chapter 14 contains no requirements corresponding to that found in Sec. 4.3.1.2.1 (related to *R* factors for systems in orthogonal directions).]

12.2.4.2 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.4}{4/1.5} = 0.15$$

From analysis, T = 0.24 seconds. The fundamental period for nonbuilding structures, is generally approximated for the first iteration and must be verified with final calculations. For many nonbuilding structures the maximum period limit contained in the first paragraph of *Provisions* Sec. 5.4.2 [5.2.2] is not appropriate. As a result, the examples in this chapter neglect that limit. Future editions of the *Provisions* are expected to clarify that this limit does not apply to nonbuilding structures. [In the 2003 *Provisions*, Sec. 14.2.9 makes clear that the approximate period equations do not apply to nonbuilding structures.]

Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed:

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.18}{0.24\left(4/1.5\right)} = 0.281$$

Provisions Sec. 14.5.1 [14.2.8] provides equations for minimum values of C_s that replace corresponding equations in *Provisions* Sec. 5.4.1.1 [5.2.1.1]. However, according to Item 2 of Sec. 14.5.1 [14.2.8, replacement of Chapter 5 equations for minima occurs only "for nonbuilding systems that have an R value provided in Table 14.5.1.1" [14.4-2]. In the present example the R values are taken from Table 5.2.2 so the minima defined in Sec. 5.4.1.1 apply. [In the 2003 *Provisions* this is no longer the case as reference to Table 4.3-1 has been eliminated. Since the example structure would satisfy exception 1 of Sec. 14.2.8 and the minimum base shear equation in Chapter 5 was removed, no additional minimum base shear must be considered.]

Using *Provisions* Eq. 5.4.1.1-3, C_s shall not be less than:

$$C_s = 0.044IS_{DS} = 0.044(1.5)(0.4) = 0.0264$$

Provisions Eq. 5.4.1.1-1 [5.2-2] controls; C_s = 0.12.

12.2.4.3 Seismic Weight

$$W = 2(240\text{ ft})(20\text{ ft})(35\text{ psf} + 10\text{ psf}) = 432\text{ kips}$$

12.2.4.4 Base Shear

Using *Provisions* Eq. 5.3.2 [5.2-1]:

$$V = C_sW = 0.15(432\text{ kips}) = 64.8\text{ kips}$$

12.2.4.5 Redundancy Factor

For the longitudinal direction, the diaphragm is the horizontal bracing in the bay with the braced frames. However, given the basis for the redundancy factor, it appears that a more appropriate definition of A_x would be the area contributing to horizontal forces in the diagonal braces. Thus A_x = 20(240) = 4800 ft^2. The ratio r_x is 0.25; each of the four braces has the same stiffness, and each is capable of tension and compression. Therefore:

$$\rho = 2 - \frac{20}{0.25\sqrt{4800}} = 0.85 < 1.0, \quad \text{use } 1.0$$

[The redundancy requirements have been changed substantially in the 2003 *Provisions*.]

12.3 STEEL STORAGE RACK, OXFORD, MISSISSIPPI

This example uses the equivalent lateral force (ELF) procedure to calculate the seismic base shear in the east-west direction for a steel storage rack.

12.3.1 Description

A four-tier, five-bay steel storage rack is located in a retail discount warehouse. There are concentrically braced frames in the north-south and east-west directions. The general public has direct access to the aisles and merchandise is stored on the upper racks. The rack is supported on a slab on grade. The design operating load for the rack contents is 125 psf on each tier. The weight of the steel support structure is assumed to be 5 psf on each tier.

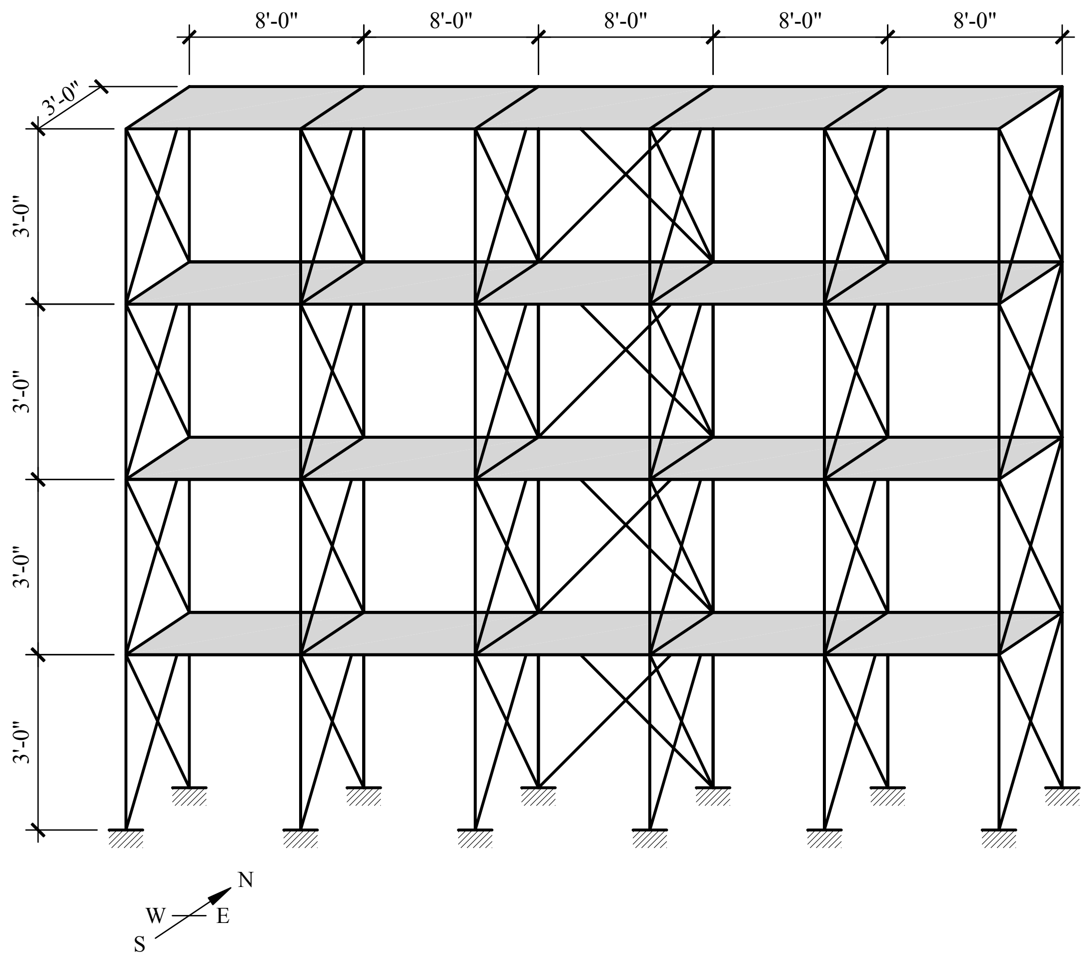


Figure 12-3 Steel storage rack (1.0 ft = 0.3048 m).

12.3.2 *Provisions* Parameters

12.3.2.1 Ground Motion

The spectral response acceleration coefficients at the site are as follows:

$S_{DS} = 0.40$
$S_{D1} = 0.18$

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package).]

12.3.2.2 Seismic Use Group and Importance Factor

Use *Provisions* Sec. 1.3 [1.2]. The storage rack is in a retail facility. Therefore the storage rack is assigned to Seismic Use Group I. According to *Provisions* Sec. 14.6.3.1 and 6.1.5 [14.3.5.2], $I = I_p = 1.5$ because the rack is in an area open to the general public.

12.3.2.3 Seismic Design Category

Use *Provisions* Tables 4.2.1a and 4.2.1b [1.4-1 and 1.4-2]. Given Seismic Use Group I, $S_{DS} = 0.40$, and $S_{D1} = 0.18$, the Seismic Design Category is C.

12.3.2.4 Design Coefficients

According to *Provisions* Table 14.5.1.1 [14.2-3], the design coefficients for this steel storage rack are

$R = 4$
$\Omega_0 = 2$
$C_d = 3½$

12.3.3 Design of the System

12.3.3.1 Seismic Response Coefficient

Provisions Sec. 14.6.3 [14.3.5]allows designers some latitude in selecting the seismic design methodology. Designers may use the Rack Manufacturer's Institute specification if they modify the equations to incorporate the seismic spectral ordinates contained in the *Provisions*; or they may use an R of 4 and use *Provisions* Chapter 5 according to the exception in *Provisions* Sec. 14.6.3.1. The exception is used in this example. [In the 2003 *Provisions* these requirements have been restructured so that the primary method is use of Chapter 5 with the design coefficients of Chapter 14; racks designed using the RMI method of Sec. 14.3.5.6 are deemed to comply.]

Using *Provisions* Eq. 5.4.1.1-1 [5.2-3]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.4}{4/1.5} = 0.15$$

From analysis, $T = 0.24$ seconds. For this particular example the short period spectral value controls the design. The period, for taller racks, however, may be significant and will be a function of the operating weight. Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.18}{0.24(4/1.5)} = 0.281$$

Provisions Sec. 14.5.1 [14.2.8]provides equations for minimum values of C_s that replace corresponding equations in Sec. 5.4.1.1 [5.2.1.1]. The equations in Sec. 14.5.1 [14.2.8] are more conservative than those in Sec. 5.4.1.1 [5.2.1.1] because nonbuilding structures generally lack redundancy and are not as highly damped as building structures. These equations generally govern the design of systems with long periods. According to Item 2 of Sec. 14.5.1 [14.2.8], replacement of the Chapter 5 equations for minima occurs only “for nonbuilding systems that have an *R* value provided in Table 14.5.1.1” [14.2-2]. In the present example the *R* value is taken from Table 14.5.1.1 [14.2-2]and the Seismic Design Category is C so Eq. 14.5.1-1 [14.2-2] applies. Using that equation, C_s shall not be less than the following:

$$C_s = 0.14 S_{DS} I = 0.14(0.4)(1.5) = 0.084$$

Provisions Eq. 5.4.1.1-1[5.2-2] controls; $C_s = 0.15$.

12.3.3.2 Condition “a” (each rack loaded)

12.3.3.2.1 Seismic Weight

In accordance with *Provisions* Sec. 14.6.3.2 [14.3.5.3], Item a:

$$W_a = 4(5)(8 \text{ ft})(3 \text{ ft})[0.67(125 \text{ psf})+5 \text{ psf}] = 42.6 \text{ kips}$$

12.3.3.2.2 Design Forces and Moments

Using *Provisions* Eq. 5.4.1 [5.2-1], the design base shear for condition “a” is calculated

$$V_a = C_s W = 0.15(42.6 \text{ kips}) = 6.39 \text{ kips}$$

In order to calculate the design forces, shears, and overturning moments at each level, seismic forces must be distributed vertically in accordance with *Provisions* Sec. 14.6.3.3 [14.3.5.4]. The calculations are shown in Table 12.3-1.

Table 12.3-1 Seismic Forces, Shears, and Overturning Moments

Level x	W_x (kips)	h_x (ft)	$w_x h_x^k$ (k = 1)	C_{vx}	F_x (kips)	V_x (kips)	M_x (ft-kips)
5	10.65	12	127.80	0.40	2.56		
						2.56	7.68
4	10.65	9	95.85	0.30	1.92		
						4.48	21.1
3	10.65	6	63.90	0.20	1.28		
						5.76	38.4
2	10.65	3	31.95	0.10	0.63		
						6.39	57.6
Σ	42.6		319.5				

1.0 ft = 0.3048 m, 1.0 kip = 4.45 kN, 1.0 ft-kip = 1.36 kN-m.

12.3.3.2.3 Resisting Moment at the Base

$M_{OT,\ resisting} = W_a$ (1.5 ft) = 42.6(1.5 ft) = 63.9 ft-kips

12.3.3.3 Condition "b" (only top rack loaded)

12.3.3.3.1 Seismic Weight

In accordance with *Provisions* Sec. 14.6.3.2 [14.3.5.3], Item b:

W_b = 1(5)(8 ft)(3 ft)(125 psf) + 4(5)(8 ft)(3 ft)(5 psf) = 17.4 kips

12.3.3.3.2 Base Shear

Using *Provisions* Eq. 5.4.1 [5.2-1], the design base shear for condition "b" is calculated as follows:

$V_b = C_s W$ = 0.15(17.4 kips) = 2.61 kips

12.3.3.3.3 Overturning Moment at the Base

Although the forces could be distributed as shown above for condition "a", a simpler, conservative approach for condition "b" is to assume that a seismic force equal to the entire base shear is applied at the top level. Using that simplifying assumption,

$M_{OT} = V_b$ (12 ft) = 2.61 kip (12 ft) = 31.3 ft-kips

12.3.3.3.4 Resisting Moment at the Base

$M_{OT,\ resisting} = W_b$ (1.5 ft) = 17.4(1.5 ft) = 26.1 ft-kips

12.3.3.4 Controlling Conditions

Condition "a" controls shear demands at all but the top level.
Although the overturning moment is larger under condition "a ," the resisting moment is larger than the overturning moment. Under condition "b" the resistance to overturning is less than the applied

overturning moment. Therefore, the rack anchors must be designed to resist the uplift induced by the base shear for condition “b”.

12.3.3.5 Torsion

It should be noted that the distribution of east-west seismic shear will induce torsion in the rack system because the east-west brace is only on the back of the storage rack. The torsion should be resisted by the north-south braces at each end of the bay where the east-west braces are placed. If the torsion were to be distributed to each end of the storage rack, the engineer would be required to calculate the transfer of torsional forces in diaphragm action in the shelving, which may be impractical.

12.4 ELECTRIC GENERATING POWER PLANT, MERNA, WYOMING

This example highlights some of the differences between the design of nonbuilding structures and the design of building structures. The boiler building in this example illustrates a solution using the equivalent lateral force (ELF) procedure. Due to mass irregularities, the boiler building would probably also require a modal analysis. For brevity, the modal analysis is not illustrated.

12.4.1 Description

Large boilers in coal-fired electric power plants are generally suspended from support steel near the roof level. Additional lateral supports (called buck stays) are provided near the bottom of the boiler. The buck stays resist lateral forces but allow the boiler to move vertically. Lateral seismic forces are resisted at the roof and at the buck stay level. Close coordination with the boiler manufacturer is required in order to determine the proper distribution of seismic forces.

In this example, a boiler building for a 950 mW coal-fired electric power generating plant is braced laterally with ordinary concentrically braced frames in both the north-south and the east-west directions. The facility is part of a grid and is not for emergency back up of a Seismic Use Group III facility.

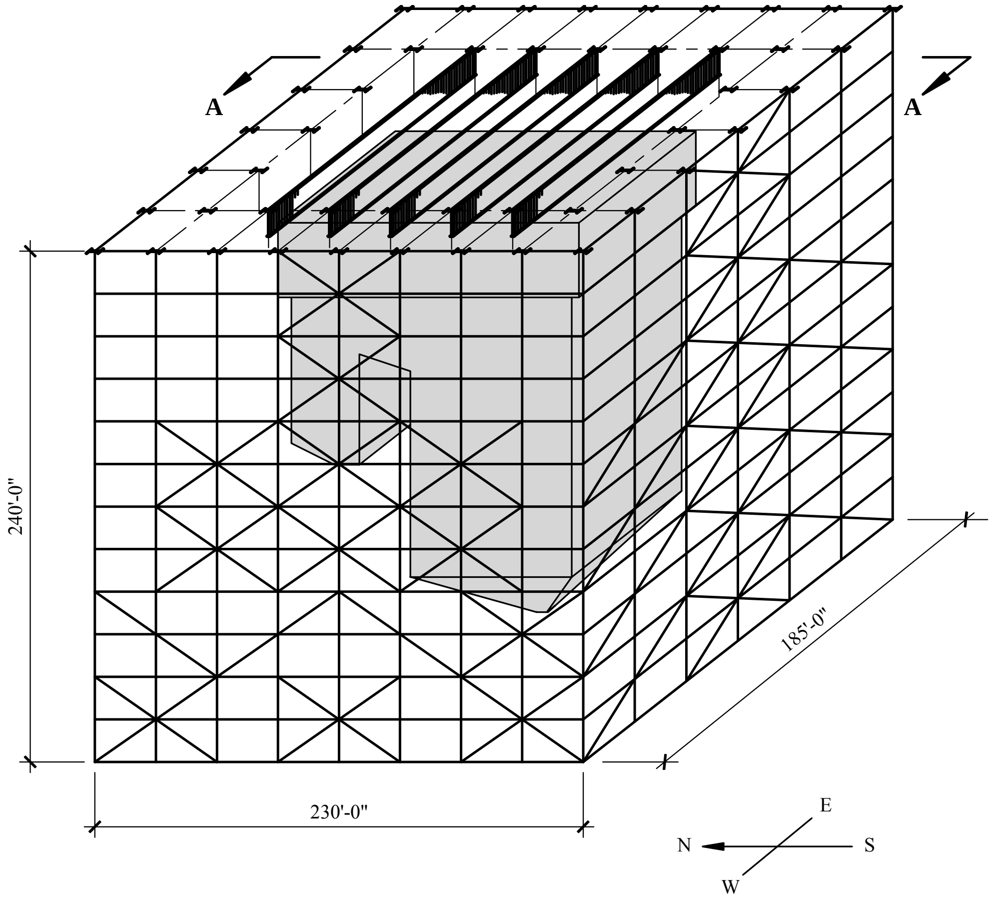
The dead load of the structure, equipment, and piping, W_{DL}, is 16,700 kips.

The weight of the boiler in service, W_{Boiler}, is 31,600 kips.

The natural period of the structure (determined from analysis) is as follows:

North-South, T_{NS} = 1.90 seconds

East-West, T_{EW} = 2.60 seconds


A
A
240'-0"
230'-0"
185'-0"
E
N
S
W

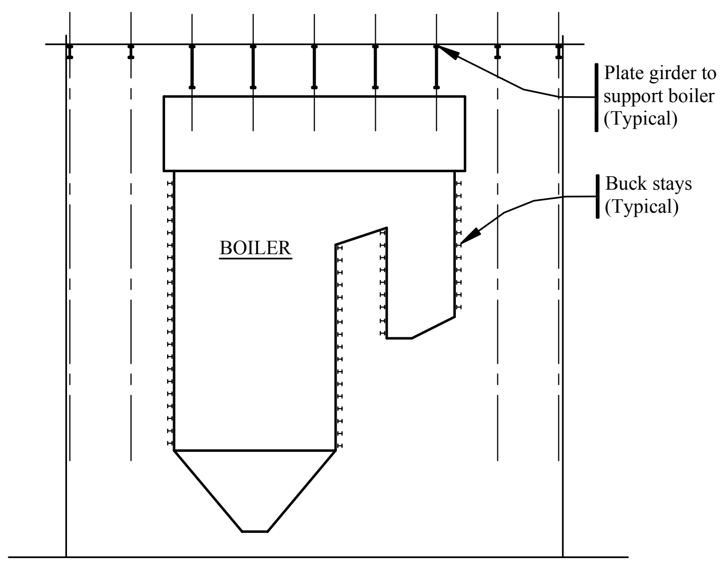


Plate girder to support boiler (Typical)
Buck stays (Typical)
BOILER

Section A-A

12.4.2 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2]) (for continuous operation, but not for emergency back up of a Seismic Use Group III facility)	=	II
Occupancy Importance Factor, I (*Provisions* Sec. 1.4 [14.2.1])	=	1.25
Site Coordinates	=	42.800° N, 110.500° W
Short Period Response, S_S (*Seismic Design Parameters*)	=	0.966
One Second Period Response, S_1 (*Seismic Design Parameters*)	=	0.278
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	=	D (default)
Acceleration-based site coefficient, F_a (*Provisions* Table 4.1.2.4a [3.3-1])	=	1.11
Velocity-based site coefficient, F_v (*Provisions* Table 4.1.2.4b [3.3-2])	=	1.84
Design spectral acceleration response parameters		
$S_{DS} = (2/3)S_{MS} = (2/3)F_aS_S = (2/3)(1.11)(0.966)$	=	0.715
$S_{D1} = (2/3)S_{M1} = (2/3)F_vS_1 = (2/3)(1.84)(0.278)$	=	0.341
Seismic Design Category (*Provisions* Sec. 4.2 [1.4])	=	D
Seismic-Force-Resisting System (*Provisions* Table 14.5.1.1 [14.2-2])	=	Steel concentrically braced frame (Ordinary)
Response Modification Coefficient, R (*Provisions* Table 5.2.2)	=	5
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2)	=	2
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2)	=	4½
Height limit (*Provisions* Table 14.5.1.1)	=	None

Note: If the structure were classified as a "building," its height would be limited to 35 ft for a Seismic Design Category D ordinary steel concentrically braced frame, according to the *Provisions* Table 5.2.2. The structure is, however, defined as a nonbuilding structure according to *Provisions* Sec. 14.6.3.4. *Provisions* Table 14.5.1.1 does not restrict the height of a nonbuilding structure using an ordinary steel concentrically braced frame.

[Changes in the 2003 *Provisions* would affect this example significantly. Table 14.2-2 would be used to determine design coefficients and corresponding levels of detailing. For structures of this height using an ordinary concentrically braced frame system, $R = 1.5$, $\Omega_0 = 1$, and $C_d = 1.5$. Alternatively, a special concentrically braced frame system could be employed.]

12.4.3 Design in the North-South Direction

12.4.3.1 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1[5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.715}{5/1.25} = 0.179$$

From analysis, T = 1.90 seconds. Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.341}{1.90\left(5/1.25\right)} = 0.045$$

but using *Provisions* Eq. 5.4.1.1-3, C_s shall not be less than:

$$C_s = 0.044IS_{DS} = 0.044(1.25)(0.715) = 0.0393$$

[Under the 2003 *Provisions* no additional minimum base shear must be considered since the example structure would satisfy exception 1 of Sec. 14.2.8 and the minimum base shear equation in Chapter 5 was removed.]

Provisions Eq. 5.4.1.1-2 [5.2-3] controls; C_s = 0.045.

12.4.3.2 Seismic Weight

Calculate the total seismic weight, W, as:

$$W = W_{DL} + W_{Boiler} = 16{,}700 \text{ kips} + 31{,}600 \text{ kips} = 48{,}300 \text{ kips}$$

12.4.3.3 Base Shear

Using *Provisions* Eq. 5.4.1 [5.2-1]:

$$V = C_sW = 0.045(48{,}300 \text{ kips}) = 2170 \text{ kips}$$

12.4.3.4 Redundancy Factor

Refer to Sec. 12.2.3.6 for an explanation of the application of this factor to nonbuilding structures similar to buildings. The seismic force resisting system is an ordinary concentric braced frame with five columns in a single line of framing. The number of bays of bracing diminishes near the top, and the overall plan area is large. For the purposes of this example, it will be assumed that the structure lacks redundancy and ρ = 1.5.

[The redundancy requirements have been substantially changed in the 2003 *Provisions*. If it is assumed that the structure would fail the redundancy criteria, ρ = 1.3.]

12.4.3.5 Determining E

See Sec. 12.2.3.7.

12.4.4 Design in the East-West Direction

12.4.4.1 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.715}{5/1.25} = 0.179$$

From analysis, T = 2.60 seconds. Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.341}{2.60(5/1.25)} = 0.0328$$

Using *Provisions* Eq. 5.4.1.1-3, C_s shall not be less than:

$$C_s = 0.044IS_{DS} = 0.044(1.25)(0.715) = 0.0393$$

[Under the 2003 *Provisions* no additional minimum base shear must be considered since the example structure would satisfy exception 1 of Sec. 14.2.8 and the minimum base shear equation in Chapter 5 was removed.]

Provisions Eq. 5.4.1.1-3 controls; C_s = 0.0393. [Under the 2003 *Provisions*, Eq. 5.2-3 would control the base shear coefficient for this example.]

12.4.4.2 Seismic Weight

Calculate the total seismic weight, W, as

$$W = W_{DL} + W_{Boiler} = 16{,}700 \text{ kips} + 31{,}600 \text{ kips} = 48{,}300 \text{ kips}$$

12.4.4.3 Base Shear

Using *Provisions* Eq. 5.4.1 [5.2-1]:

$$V = C_sW = 0.0393(48{,}300 \text{ kips}) = 1900 \text{ kips}$$

12.5 PIER/WHARF DESIGN, LONG BEACH, CALIFORNIA

This example illustrates the calculation of the seismic base shear in the east-west direction for the pier using the ELF procedure.

12.5.1 Description

A private shipping company is developing a pier in Long Beach, California, to service container vessels. In the north-south direction, the pier is tied directly to an abutment structure supported on grade. In the east-west direction, the pier resists seismic forces using moment frames.

The design live load for container storage is 1000 psf.

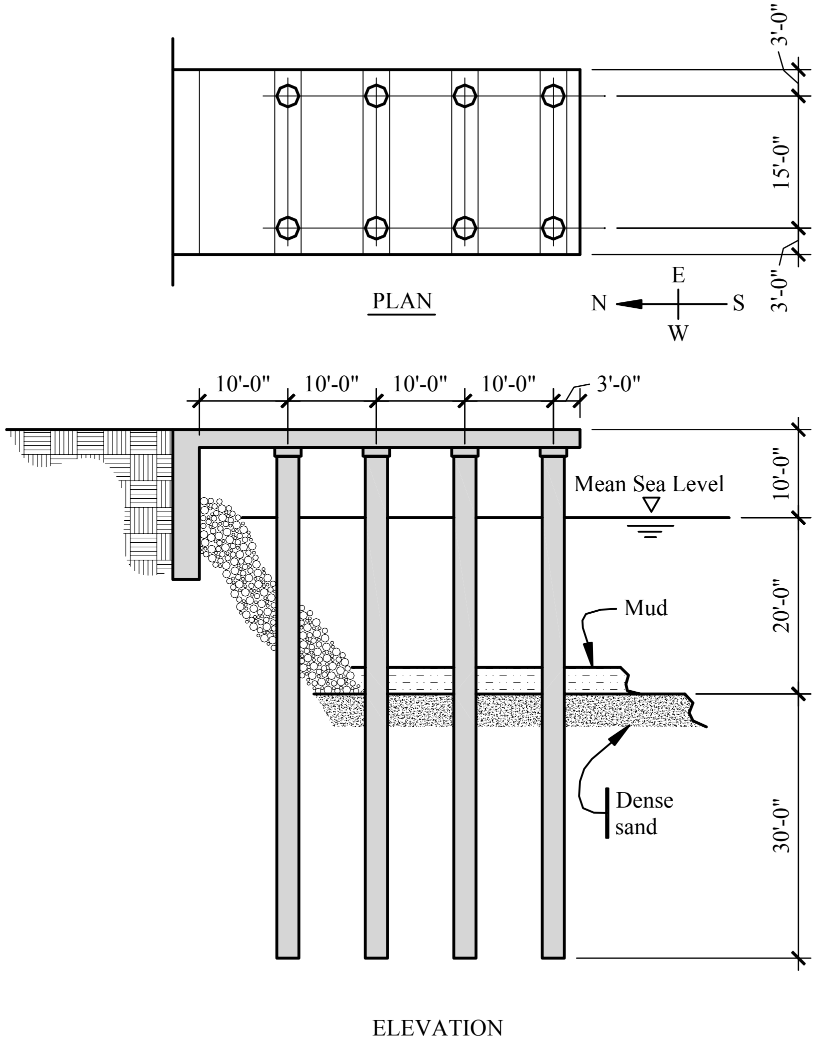


Figure 12-5 Pier plan and elevation (1.0 ft = 0.3048 m).

12.5.2 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2]) (The pier serves container vessels that carry no hazardous materials.)	=	I
Importance Factor, I (*Provisions* Sec. 14.5.1.2 [14.2.1])	=	1.0
Short Period Response, S_S	=	1.75
One Second Period Response, S_1	=	0.60
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	=	D (dense sand)
Acceleration-based Site Coefficient, F_a (*Provisions* Table 4.1.2.4a [3.3-1])	=	1.0

Velocity-based Site Coefficient, F_v (*Provisions* Table 4.1.2.4b [3.3-2])	=	1.5

Design spectral acceleration response parameters

$S_{DS} = (2/3)S_{MS} = (2/3)F_aS_S = (2/3)(1.0)(1.75)$	=	1.167
$S_{D1} = (2/3)S_{M1} = (2/3)F_vS_1 = (2/3)(1.5)(0.60)$	=	0.60
Seismic Design Category (*Provisions* Sec. 4.2)	=	D
Seismic-Force-Resisting System (*Provisions* Table 14.5.1.1 [14.2-2])	=	Intermediate concrete moment frame
Response Modification Coefficient, R (*Provisions* Table 5.2.2)	=	5

(The *International Building Code* and the 2002 edition of ASCE 7 would require an R value of 3.)

System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2)	=	3
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2)	=	4½
Height limit (*Provisions* Table 14.5.1.1)	=	50 ft

If the structure was classified as a building, an intermediate reinforced concrete moment frame would not be permitted in Seismic Design Category D.

[Changes in the 2003 *Provisions* would affect this example significantly. Table 14.2-2 would be used to determine design coefficients and corresponding levels of detailing. For structures of this height using an intermediate concrete moment frame system, $R = 3$, $\Omega_0 = 2$, and $C_d = 2.5$.]

12.5.3 Design of the System

12.5.3.1 Seismic Response Coefficient

Using *Provisions* Eq. 5.4.1.1-1 [5.2-2]:

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.167}{5/1.0} = 0.233$$

From analysis, $T = 0.596$ seconds. Using *Provisions* Eq. 5.4.1.1-2 [5.2-3], C_S does not need to exceed:

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.60}{0.596(5/1.0)} = 0.201$$

Using *Provisions* Eq. 5.4.1.1-3 , C_s shall not be less than:

$C_s = 0.044IS_{DS} = 0.044(1.0)(1.167) = 0.0513$

[Under the 2003 *Provisions* no additional minimum base shear must be considered since the example structure would satisfy exception 1 of Sec. 14.2.8 and the minimum base shear equation in Chapter 5 was removed.]

Provisions Eq. 5.4.1.1-2 [5.2-3] controls; $C_s = 0.201$.

12.5.3.2 Seismic Weight

In accordance with Provisions Sec. 5.3 [5.2.1] and 14.6.6 [14.2.6], calculate the dead load due to the deck, beams, and support piers, as follows:

$$W_{Deck} = 1.0\text{ ft}(43\text{ ft})(21\text{ ft})(0.150\text{ kip/ft}^3) = 135.5\text{ kips}$$

$$W_{Beam} = 4(2\text{ ft})(2\text{ ft})(21\text{ ft})(0.150\text{ kip/ft}^3) = 50.4\text{ kips}$$

$$W_{Pier} = 8[\pi(1.25\text{ ft})^2][(10\text{ ft} - 3\text{ ft}) + (20\text{ ft})/2](0.150\text{ kip/ft}^3) = 100.1\text{ kips}$$

$$W_{DL} = W_{Deck} + W_{Beams} + W_{Piers} = 135.5 + 50.4 + 100.1 = 286.0\text{ kips}$$

Calculate 25 percent of the storage live load

$$W_{1/4\,LL} = 0.25(1000\text{ psf})(43\text{ ft})(21\text{ ft}) = 225.8\text{ kips}$$

Calculate the weight of the displaced water (*Provisions* Sec. 14.6.6 [14.3.3.1])

$$W_{Disp.\ water} = 8[\pi(1.25\text{ ft})^2](20\text{ ft})(64\text{ pcf}) = 50.27\text{ kips}$$

Therefore, the total seismic weight is

$$W = W_{DL} + W_{1/4LL} + W_{Disp.\ water} = 286.0 + 225.8 + 50.27 = 562.1\text{ kips}$$

12.5.3.3 Base Shear

Using *Provisions* Eq. 5.3.2 [5.2-1]:

$$V = C_sW = 0.201(562.1\text{ kips}) = 113.0\text{ kips}$$

12.5.3.4 Redundancy Factor

This structure is small in area and has a large number of piles. Following the method described in Sec. 12.2.3.6, yields $\rho = 1.0$.

12.6 TANKS AND VESSELS, EVERETT, WASHINGTON

The seismic response of tanks and vessels can be significantly different from that of buildings. For a structure composed of interconnected solid elements, it is not difficult to recognize how ground motions accelerate the structure and cause inert forces within the structure. Tanks and vessels, when empty, respond in a similar manner.

When there is liquid in the tank, the response is much more complicated. As earthquake ground motions accelerate the tank shell, the shell applies lateral forces to the liquid. The liquid, which responds to those lateral forces. The liquid response may be amplified significantly if the period content of the earthquake ground motion is similar to the natural sloshing period of the liquid.

Earthquake-induced impulsive fluid forces are those calculated assuming that the liquid is a solid mass. The convective fluid forces are those that result from sloshing in the tank. It is important to account for the convective forces on columns and appurtenances inside the tank, because they are affected by sloshing in the same way that waves affect a pier in the ocean.

The freeboard considerations are critical. Often times, the roof acts as a structural diaphragm. If a tank does not have sufficient freeboard, the sloshing wave can rip the roof from the wall of the tank. This could result in the failure of the wall and loss of the liquid within.

The nature of seismic design for liquid containing tanks and vessels is complicated. The fluid mass that is effective for impulsive and convective seismic forces is discussed in the literature referenced in the NEHRP *Provisions* and *Commentary*.

12.6.1 Flat-Bottom Water Storage Tank

12.6.1.1 Description

This example illustrates the calculation of the design base shear using the equivalent lateral force (ELF) procedure for a steel water storage tank used to store potable water for a process within a chemical plant (Figure 12-6).

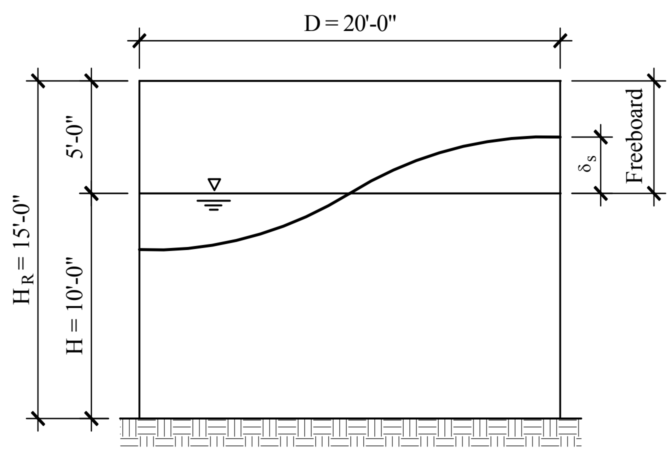


Figure 12-6 Storage tank section (1.0 ft = 0.3048 m).

The tank is located away from personnel working within the facility.

The weight of the tank shell, roof, and equipment is 15,400 lb.

12.6.1.2 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2])	=	I
Importance Factor, I (*Provisions* Sec. 14.5.1.2 [14.2.1])	=	1.0
Site Coordinates	=	48.000° N, 122.250° W
Short Period Response, S_S	=	1.236

One Second Period Response, S_1	=	0.406
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	=	C (per geotech)
Acceleration-based Site Coefficient, F_a (*Provisions* Table 4.1.2.4a [3.3-1])	=	1.0
Velocity-based Site Coefficient, F_v (*Provisions* Table 4.1.2.4b [3.3-2])	=	1.39

Design spectral acceleration response parameters

$S_{DS} = (2/3)S_{MS} = (2/3)F_aS_S = (2/3)(1.0)(1.236)$	=	0.824
$S_{D1} = (2/3)S_{M1} = (2/3)F_vS_1 = (2/3)(1.39)(0.406)$	=	0.376
Seismic-Force-Resisting System (*Provisions* Table 14.5.1.1 [14.2-3])	=	Flat-bottom, ground-supported, anchored, bolted steel tank
Response Modification Coefficient, R (*Provisions* Table 14.5.1.1 [14.2-3])	=	3
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [14.2-3])	=	2
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 [14.2-3])	=	2½

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

12.6.1.3 Calculations for Impulsive Response

12.6.1.3.1 Natural Period for the First Mode of Vibration

Based on analysis, the period for impulsive response of the tank and its contents is T_i = 0.14 sec.

12.6.1.3.2 Spectral Acceleration

Based on *Provisions* Figure 14.7.3.6-1 [14.4-1]:

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.376}{0.824} = 0.456 \text{ seconds}$$

Using *Provisions* Sec. 14.7.3.6.1 [114.4.7.5.1] with $T_i < T_s$,:

$$S_{ai} = S_{DS} = 0.824$$

12.6.1.3.3 Seismic (Impulsive) Weight

W_{tank} = 15.4 kips

$W_{1water} = \pi(10\text{ ft})^2(10\text{ ft})(0.0624\text{ kip/ft}^3)\ (W_1/W_T) = 196.0\ (0.75)$ kips = 147 kips

The ratio W_1/W_T (= 0.75) was determined from AWWA D100 (it depends on the ratio of height to diameter)

$W_i = W_{tank} + W_{1water}$ = 15.4 + 147 = 162.4 kips

12.6.1.3.4 Base Shear

According to *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1]:

$$V_i = \frac{S_{ai}W_i}{R} = \frac{0.824(162.4\text{ kips})}{3} = 44.6\text{ kips}$$

12.6.1.4 Calculations for Convective Response Natural Period for the First Mode of Sloshing

12.6.1.4.1 Natural Period for the First Mode of Sloshing

Using *Provisions* Section 14.7.3.6.1 [14.4.7.5.1]:

$$T_c = 2\pi\sqrt{\frac{D}{3.68g\tanh\left(\frac{3.68H}{D}\right)}} = 2\pi\sqrt{\frac{20\text{ ft}}{3.68\left(32.174\,\frac{\text{ft}}{\text{s}^2}\right)\tanh\left(\frac{3.68(10\text{ ft})}{10\text{ ft}}\right)}} = 2.58\text{ s}$$

12.6.1.4.2 Spectral Acceleration

Using *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1] with T_c < 4 seconds:

$$S_{ac} = \frac{1.5S_{D1}}{T_c} = \frac{1.5(0.376)}{2.58} = 0.219$$

12.6.1.4.3 Seismic (Convective) Weight

$W_c = W_{water}\ (W_2/W_T) = 196\ (0.30)$ = 58.8 kips

The ratio W_2/W_T (= 0.30) was determined from AWWA D100.

12.6.1.4.4 Base Shear

According to *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1]:

$$V_c = \frac{S_{ac}W_c}{R} = \frac{0.219(58.8\text{ kips})}{3} = 4.29\text{ kips}$$

12.6.1.5 Design Base Shear

Although Item b of *Provisions* Sec. 14.7.3.2 [14.4.7.1] indicates that impulsive and convective components may, in general, be combined using the SRSS method, *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1] requires that the direct sum be used for ground-supported storage tanks for liquids. Using *Provisions* Eq. 14.7.3.6.1 [14.4-1]:

$V = V_i + V_c = 44.6 + 4.29 = 48.9$ kips

[In the 2003 *Provisions*, use of the SRSS method is also permitted for ground-supported storage tanks for liquids.]

12.6.2 FLAT-BOTTOM GASOLINE TANK

12.6.2.1 Description

This example illustrates the calculation of the base shear and the required freeboard using the ELF procedure for a petro-chemical storage tank in a refinery tank farm near a populated city neighborhood. An impoundment dike is not provided to control liquid spills.

The tank is a flat-bottom, ground-supported, anchored, bolted steel tank constructed in accordance with API 650. The weight of the tank shell, roof, and equipment is 15,400 lb.

12.6.2.2 *Provisions* Parameters

Seismic Use Group (*Provisions* Sec. 1.3 [1.2])	=	III
(The tank is used for storage of hazardous material.)		
Importance Factor, I (*Provisions* Sec. 14.5.1.2 [14.2.1])	=	1.5
Site Coordinates	=	48.000° N, 122.250° W
Short Period Response, S_S	=	1.236
One Second Period Response, S_1	=	0.406
Site Class (*Provisions* Sec. 4.1.2.1 [3.5])	=	C (per geotech)
Acceleration-based Site Coefficient, F_a (*Provisions* Table 4.1.2.4a [3.3-1])	=	1.0
Velocity-based Site Coefficient, F_v (*Provisions* Table 4.1.2.4b [3.3-2])	=	1.39

Design spectral acceleration response parameters

$S_{DS} = (2/3)S_{MS} = (2/3)F_aS_S = (2/3)(1.0)(1.236) = 0.824$
$S_{D1} = (2/3)S_{M1} = (2/3)F_vS_1 = (2/3)(1.39)(0.406) = 0.376$

Seismic-Force-Resisting System (*Provisions* Table 14.5.1.1 [14.2-3])	=	Flat-bottom, ground-supported, anchored, bolted steel tank
Response Modification Coefficient, R (*Provisions* Table 14.5.1.1 [14.2-3])	=	3
System Overstrength Factor, Ω_0 (*Provisions* Table 5.2.2 [14.2-3])	=	2
Deflection Amplification Factor, C_d (*Provisions* Table 5.2.2 [14.2-3])	=	2½

[The 2003 *Provisions* have adopted the 2002 USGS probabilistic seismic hazard maps, and the maps have been added to the body of the 2003 *Provisions* as figures in Chapter 3 (instead of the previously used separate map package). The CD-ROM also has been updated.]

12.6.2.3 Calculations for Impulsive Response

12.6.2.3.1 Natural Period for the First Mode of Vibration

Based on analysis, the period for impulsive response of the tank and its contents is T_i = 0.14 sec.

12.6.2.3.2 Spectral Acceleration

Based on *Provisions* Figure 14.7.3.6-1 [14.4-1]:

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.376}{0.824} = 0.456 \text{ seconds}$$

Using *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1] with $T_i < T_s$,:

$S_{ai} = S_{DS} = 0.824$

12.6.2.3.3 Seismic (Impulsive) Weight

W_{tank} = 15.4 kips

$W_{Gas} = \pi(10\text{ ft})^2(10\text{ ft})(0.046\text{ kip/ft}^3)(W_I/W_T)$ = 144.5 kips (0.75) = 108.4 kips

Note: The ratio W_I/W_T was determined from AWWA D100, but API 650 should be used.

$W_i = W_{tank} + W_{Gas}$ = 15.4 + 108.4 = 123.8 kips

12.6.2.3.4 Base Shear

According to *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1]:

$$V_i = \frac{S_{ai} I W_i}{R} = \frac{0.824(1.5)(123.8\text{ kips})}{3} = 51.0\text{ kips}$$

12.6.2.4 Calculations for Convective Response

12.6.2.4.1 Natural Period for the First Mode of Sloshing

The dimensions are the same as those used for the water tank in Sec. 12.6.1; therefore, T_c = 2.58 sec.

12.6.2.4.2 Spectral Acceleration

Likewise, S_{ac} = 0.219.

12.6.2.4.3 Seismic (Convective) Weight

$$W_c = W_{LNG}\,(W_2/W_T) = 144.5\,(\,0.30) = 43.4 \text{ kips}$$

The ratio W_2/W_T was determined from AWWA D100.

12.6.2.4.4 Base shear

According to *Provisions* Sec. 14.7.3.6.1 [14.4.7.5.1]:

$$V_c = \frac{S_{ac}IW_c}{R} = \frac{0.824(1.5)(43.5\text{ kips})}{3} = 17.9\text{ kips}$$

12.6.2.5 Design Base Shear

Using *Provisions* Eq. 14.7.3.6.1 [14.4-1]:

$$V = V_i + V_c = 51.0 + 17.9 = 68.9 \text{ kips}$$

12.6.2.6 Minimum Freeboard

Provisions Table 14.7.3.6.1.2 [14.4-2] indicates that a minimum freeboard equal to δ_s is required for this tank. Using *Provisions* Eq. 14.7.3.6.1.2 [14.4-9]:

$$\delta_s = 0.5DIS_{ac} = 0.5(20\text{ ft})(1.5)(0.219) = 3.29\text{ ft}$$

The 5 ft freeboard provided is adequate.

12.7 EMERGENCY ELECTRIC POWER SUBSTATION STRUCTURE, ASHPORT, TENNESSEE

The main section addressing electrical transmission, substation, and distribution structures is in the appendix to Chapter 14 of the *Provisions*. The information is in an appendix so that designers can take time to evaluate and comment on the seismic design procedures before they are included in the main text of the *Provisions*.

[In the 2003 *Provisions* Sections A14.2.1 and A14.2.2 were removed because the appropriate industry standards had been updated to include seismic design criteria and earthquake ground motions consistent with the *Provisions*. Therefore, all references to the *Provisions* in Sec. 12.7 of this chapter are obsolete.]

12.7.1 Description

This example illustrates the calculation of the base shear using the ELF procedure for a braced frame that supports a large transformer (Figure 12-7). The substation is intended to provide emergency electric power to the emergency control center for the fire and police departments of a community. There is only one center designed for this purpose.

The weight of the transformer equipment is 17,300 lb.

The weight of the support structure is 12,400 lb.

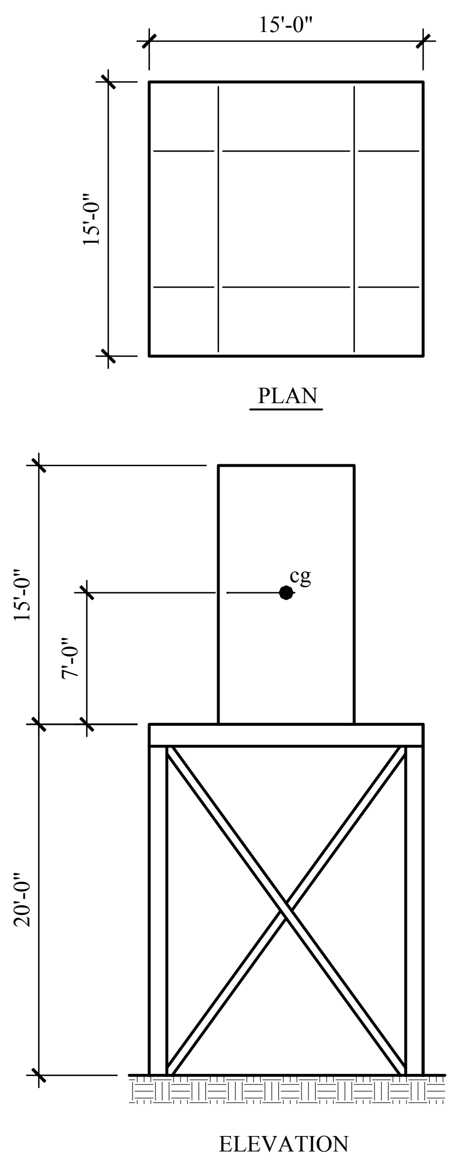


Figure 12-7 Platform for elevated transformer (1.0 ft = 0.3048 m).

The period of the structure is $T = 0.240$ sec.

Although the ratio of the supported structure over the total weight is greater than 25 percent, experience indicates that the transformer will behave as a lumped rigid mass.

12.7.2 *Provisions* Parameters

12.7.2.1 Ground Motion

The design response spectral accelerations are defined as

$S_{DS} = 1.86$
$S_{D1} = 0.79$

12.7.2.2 Seismic Use Group and Importance Factor

The structure is for emergency electric power for a Seismic Use Group III facility. Therefore, the platform is assigned to Seismic Use Group III, as required by *Provisions* Sec. 1.3 [1.2]. Using *Provisions* Table 14.5.1.2 [14.2-1], the Importance Factor, I, is equal to 1.5.

12.7.2.3 Response Modification Coefficient

From *Provisions* Table 14A.2.1, R is 3.

12.7.3 Design of the System

12.7.3.1 Seismic Response Coefficient

Provisions Sec. 14A.2.2 defines C_s in a manner that is not consistent with the rest of the *Provisions*. This inconsistency will be eliminated in future editions of the *Provisions*. In this example, the equations are applied in a manner that is consistent with Chapters 5 [4 and 5] and 14 – that is, R is applied in the calculation of C_s rather than in the calculation of V.

Using *Provisions* Section 14A.2.2:

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.86}{3/1.5} = 0.93$$

but C_s need not be larger than:

$$C_s = \frac{S_{D1}}{T\left(R/I\right)} = \frac{0.79}{0.24\left(3/1.5\right)} = 1.646$$

Therefore, $C_s = 0.93$.

12.7.3.2 Seismic Weight

$$W = W_{Transformer} + W_{Support\ structure} = 17.3 + 12.4 = 29.7 \text{ kips}$$

12.7.3.3 Base Shear

Using *Provisions* Section 14A.2.2: $V = C_sW = 0.93(29.7 \text{ kips}) = 27.6$ kips