E Development of the Design Approach to Conform with ASCE/SEI 7 and Suggested Conversion Factor Adjustments for Locations Outside the United States

This annex is not a part of the requirements of this NFPA document but is included for informational purposes only.

E.1 General.

Seismic design of nonstructural components is governed by the provisions of Chapter 13 of ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. In ASCE/SEI 7, fire sprinkler piping is classified as a "Designated Seismic System," due to its critical safety function. Design earthquake forces are multiplied by an importance factor, I_p = 1.5, and both the bracing and the piping itself must be designed for seismic forces. The seismic design requirements for the hanging and bracing of sprinkler piping systems provided in Chapter 18 of NFPA 13 presume that sprinkler piping is being constructed in the United States or in a country or jurisdictions where the seismic requirements are those specified in ASCE/SEI 7. There are locations outside the United States that wish to use the seismic design requirements of Chapter 18 of NFPA 13. In Section E.3, suggested conversion factor adjustments are provided to adjust country building code design ground motion criteria to those in ASCE/SEI 7 so the procedures of Chapter 18 of NFPA 13 can be used.

The lateral sway bracing provisions of 18.5.5 were developed to allow the use of the concept of zone of influence (ZOI), while providing designs that comply with ASCE/SEI 7. One of the main changes between the current seismic sway bracing design approach adopted in NFPA 13 and the approach used in early editions of NFPA 13 is that the spacing of the sway braces can be constrained by the flexural capacity of the pipe, as well as the capacity of the brace assembly or the capacity of the connection between the brace assembly and the supporting structure. NFPA 13 provides a design that complies with the seismic design requirements of ASCE/SEI 7 for the pipe itself.

The ZOI approach yields the force demand on the bracing element and connections to the structure. Another way to look at a ZOI force is as a reaction in a system of continuous beams (i.e., the multiple spans of a piping system). By using conservative simplifying assumptions, a maximum ZOI force limited by the flexural capacity of the pipe can be developed for a given pipe size and span (spacing between horizontal sway braces). The method used to develop these maximum ZOI forces is described in the following paragraphs, along with a discussion of the assumptions on the geometry of the piping system, the determination of the seismic design force coefficients, and the flexural capacity of the pipe.

In the discussion that follows, the term "main" can be taken to mean a sprinkler main, either a feed main or a cross-main, that requires sway bracing.

E.2 Assumptions on System Geometry.

While every fire sprinkler system is uniquely designed for a particular structure, there are general similarities in the layout and geometry that can be used to simplify the design approach for earthquake protection. These similarities were used to develop assumptions on the effects of piping system continuity on the distribution of bending and shear forces in the pipe, and assumptions on spacing of branch lines between sway brace locations.

E.2.1 Continuity in Piping Systems.

For lateral brace design purposes, piping systems can be idealized as a system of continuous beams. The bending moments in the sprinkler mains (the beams) were computed assuming three continuous spans, which generates the largest bending moment in any system of continuous beams. The loads generated by the branch lines are idealized as point loads. The tributary mass of the main is lumped along with the mass of the branch lines as point loads at the assumed branch line locations.

E.2.2 Branch Line Locations.

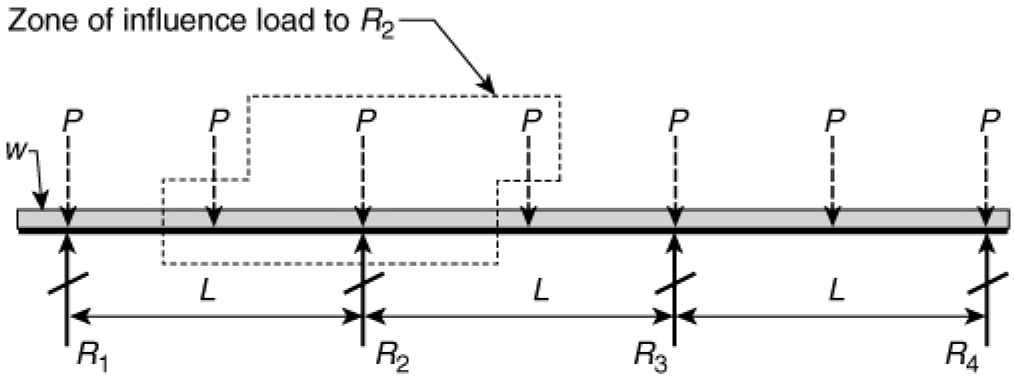
In many sprinkler system installations, the branch lines constitute a substantial portion of the seismic mass. While there are significant variations in the spacing of the branch lines, their geometry is constrained by the need to provide adequate water coverage, which imposes limits on the spacing of the branches. Defining a "span" of the main as the distance between lateral sway braces, the seismic provisions make the following assumptions:

(1) There is a branch located at the center of the sprinkler main for spans of 25 ft (7.6 m) or less.
(2) There are branches at third-points of the sprinkler main for spans greater than 25 ft (7.6 m) and less than 40 ft (12 m).
(3) There are branches at quarter-points of the sprinkler main for spans of 40 ft (12 m).

It was further assumed that there is a branch line located in close proximity to each sway brace.

The layout of branch lines, maximum bending moment M_{max} in the pipe, and reaction R_{max} (horizontal loads at sway brace locations) for sprinkler mains with spans less than 25 ft (7.6 m) is illustrated in Figure E.2.2(a). Maximum demands for spans greater than 25 ft (7.6 m) and less than 40 ft (12 m) are given in Figure E.2.2(b), and for spans of 40 ft (12 m) in Figure E.2.2(c).

Figure E.2.2(a) Maximum Demands for Spans Less Than 25 ft (7.6 m).



L = distance between sway braces (span)

P = branch line lateral load + tributary lateral load from main

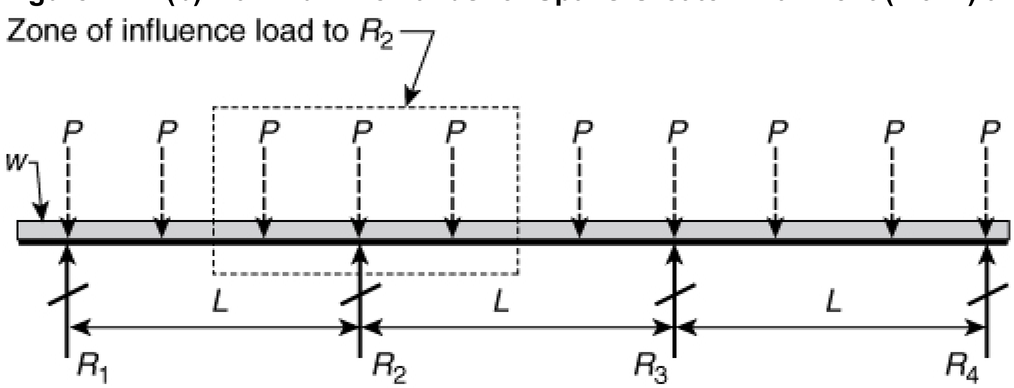
w = lateral load of the main (included in P)

R_1, R_2, R_3, R_4 = zone of influence load (reactions)

$$M_{max} = 0.175PL$$

$$R_{max} \approx 2P$$

Figure E.2.2(b) Maximum Demands for Spans Greater Than 25 ft (7.6 m) and Less Than 40 ft (12.2 m).



L = distance between sway braces (span)

P = branch line lateral load + tributary lateral load from main

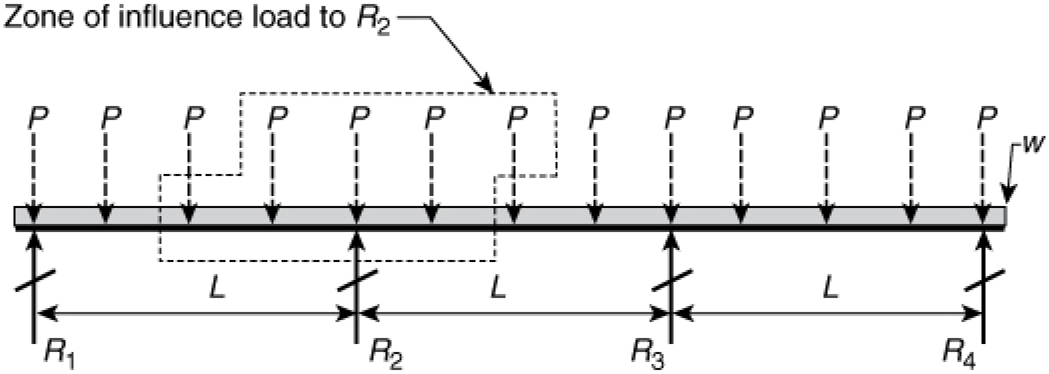
w = lateral load of the main (included in P)

R_1, R_2, R_3, R_4 = zone of influence load (reactions)

$$M_{max} = 0.267PL$$

$$R_{max} \approx 3P$$

Figure E.2.2(c) Maximum Demands for Spans of 40 ft (12 m).



L = distance between sway braces (span)
P = branch line lateral load + tributary lateral load from main
w = lateral load of the main (included in P)
R_1, R_2, R_3, R_4 = zone of influence load (reactions)

$$M_{max} = 0.372PL$$

$$R_{max} \approx 4P$$

E.3 Computing the Seismic Demand on Piping Systems.

In ASCE/SEI 7, seismic demands on nonstructural components and systems are a function of the ground shaking intensity, the ductility and dynamic properties of the component(s) or system, the ductility of the structure, and the height of attachment of the component(s) in the structure. Seismic forces are determined at strength design (SD) levels. The horizontal seismic design force is given by Equation E.3a:

$$Fp = 0.4S_{DS}I_pW_p\left[\frac{H_f}{R_\mu}\right]\left[\frac{C_{AR}}{R_{po}}\right] \qquad \text{[E.3a]}$$

where:

F_p = seismic design force

S_{DS} = short-period design spectral acceleration, which takes into account soil conditions at the site

I_p = component importance factor, taken as 1.5 for fire sprinkler systems

W_p = component operating weight

H_f = factor for force amplification as a function of height in the structure, which, when the fundamental period of the building is unknown can be taken as $[1 + 2.5(z/h)]$ where:

z = height of the component attachment to the structure with respect to the grade plane

h = average roof height of the structure with respect to the grade plane

R_μ = structure ductility reduction factor, which, when the lateral force-resisting system and structural importance factor are unknown, can be taken as 1.3 for piping supported above grade

C_{AR} = component resonance ductility factor that converts the peak floor or ground acceleration into the peak component acceleration as follows: (1) 1.0 for rigid or cable braces; (2) 1.0 for piping in accordance with ASME B31 and joints made by welding or brazing; (3) 1.0 for high- or limited deformability piping in accordance with ASME B31, *Standards of Pressure Piping*, with joints made by threading, bonding, compression couplings, or grooved couplings, or for high-deformability non-ASME B31 piping with joints made by welding or brazing; (4) 2.2 for high- or limited-deformability non-ASME B31 piping with joints made by threading, bonding, compression couplings, or grooved couplings; or (5) 2.2 for low-deformability piping such as cast iron and nonductile plastics

R_{po} = component strength factor as follows: (1) 1.5 for rigid or cable braces; (2) 3.0 for piping in accordance with ASME B31 and joints made by welding or brazing; (3) 2.0 for high- or limited deformability piping in accordance with ASME B31 with joints made by threading, bonding, compression couplings, or grooved couplings, or for high-deformability non-ASME B31 piping with joints made by welding or brazing; (4) 2.0 for high- or limited-deformability non-ASME B31 piping with joints made by threading, bonding, compression couplings, or grooved couplings; or (5) 1.5 for low-deformability piping such as cast iron and nonductile plastics

F_p need not be greater than 1.6 $S_{DS}I_pW_p$ and cannot be less than 0.30 $S_{DS}I_pW_p$.

As illustrated in Figure E.3, NFPA 13 uses a default seismic factor, C_p, which combines ground shaking, S_{DS}; component importance factor, I_p; location in the building, H_f (based on z/h); structure ductility reduction factor, R_u; component resonance ductility factor, C_{AR}; component strength factor, R_{po}; and the load factor needed to convert from strength design to allowable stress design (ASD) into a single variable. Conservative assumptions are made for each variable so that the only information needed to find C_p is the short-period design spectral acceleration for the maximum considered earthquake (MCE), S_{DS}, for the code-default soil class or the verified soil class for the location.

Figure E.3 Default Seismic Factor, C_p.

$$F_p = \boxed{\frac{0.4\, a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right)} W_p$$

$$F_{pw} = \boxed{C_p} * W_p$$

The importance factor, I_p, for fire sprinkler systems is specified in ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, as 1.5. It was assumed that the system is installed at the roof level, h (i.e., $z = h$), resulting in H_f = [1 + 2.5(z/h)] = 3.5. The minimum value of the structure ductility reduction factor, R_μ, was taken as 1.3 assuming the lateral force-resisting system and structural importance factor are unknown. Finally, since F_{pw} represents the force on braces, the component resonance ductility factor, C_{AR}, was taken as 1.0, and the component strength factor, R_{po}, was taken as 1.5.

Substitute these values into the lateral force Equation E.3b as follows:

$$F_p = 0.4 S_{DS} I_p W_p \left[\frac{H_f}{R_\mu}\right]\left[\frac{C_{AR}}{R_{po}}\right] = 0.4 S_{DS}(1.5) W_p \left[\frac{3.5}{1.3}\right]\left[\frac{1.0}{1.5}\right] = (1.077) S_{DS}\, W_p \qquad \text{[E.3b]}$$

ASCE/SEI 7 forces are determined at the SD level. NFPA 13 is based on allowable stress design (ASD). To convert F_p to an ASD load, F_{pw}, the load from ASCE/SEI 7 is multiplied by a 0.7 load factor as follows in Equation E.3c:

$$F_{pw} = 0.7 F_p = 0.7(1.077) S_{DS} W_p = 0.754\ S_{DS} W_p = C_p W_p \qquad \text{[E.3c]}$$

Solve for C_p as follows in Equation E.3d:

$$C_p = 0.754 S_{DS} \qquad \text{[E.3d]}$$

The short-period design spectral acceleration, S_{DS}, is obtained by first, modifying the short-period spectral acceleration for reference rock site conditions [site class BC — see Table E.3(a)], S_S, for the effects of the local soil conditions, and second, taking two-thirds of this soil-adjusted value. In the United States, seismic hazard maps used in ASCE/SEI 7 are based on data published by the US Geological Survey (USGS). For ASCE/SEI 7 (2016) the parameter S_S was mapped. S_S was then multiplied by a factor, *Fa*, determined based on the code default site soil class (site class C or D, whichever resulted in the most critical amplification) or on the actual site soil conditions if known, to obtain the parameter S_{MS}, which was in turn multiplied by a factor of 2/3 to determine S_{DS}. By contrast, in ASCE/SEI (2022) the parameter S_{MS} for the code default site soil class is mapped directly (in this case, using the most critical amplification of site classes C, CD, and D). To ensure accuracy and because effects of the site soil class in ASCE/SEI 7 can no longer be manually calculated, it is necessary to use the web-based ASCE/SEI 7 hazard tool, or a similar tool or site-specific procedures, to generate values for S_{DS} based on the latitude and longitude of the project site. The code default site soil class should be assumed unless actual site soil conditions are known based on data acceptable to the authority having jurisdiction (e.g., from a geotechnical report).

Most countries do not base their seismic hazard maps on the ground motion criteria that USGS uses to determine the S_{DS} values specified in ASCE/SEI 7. Instead, these countries might use seismic zones [similar to those in the outdated *Uniform Building Code* (UBC)] to convey the seismic hazard. Although different countries might use different zone identifiers, zones are often numbered with the highest number seismic zone having the strongest potential ground motions (e.g., in the UBC, Zones 0 to 4 were used, and Zone 4 had the highest seismic hazard). Although not universally true, there is often a zone factor, Z, associated with each zone that represents the peak ground acceleration (sometimes adjusted for near faults and other amplifications) based on design earthquake ground motions having a 10 percent chance of being exceeded in a 50-year period (i.e., about a 500-year return period). For these countries, a suggested correlating adjustment is S_{DS} = 2.5 Z without other adjustments considered. For example, for a Z factor (PGA) of 0.4 (i.e., the highest value in the UBC without adjustment), the value of S_{DS} would be 1.0, (resulting in C_p = 0.754 multiplier for braces in 18.5.9.3). If a value of the one-second spectral acceleration for reference rock site conditions, S_{D1}, is needed for these countries, the value might be taken as 1.5 Z, which is the same relative relationship between the short-period and one-second spectral acceleration that was used in the 1997 UBC. The spectral acceleration used for seismic design, S_{DS}, is determined by Equation E.3e:

$$S_{DS} = \frac{2}{3} S_S F_a \qquad \text{[E.3e]}$$

F_a is an amplification factor based on soil conditions and the intensity of ground shaking expected, measured by S_S. Soil conditions are defined by site class in Table 20.3-1 of ASCE/SEI 7 (2016) or Table 20.2-1 of ASCE/SEI 7 (2022). The ASCE/SEI 7 (2022) site classes are summarized in Table E.3(a) of NFPA 13. The values of F_a are given in Table 11.4-1 of ASCE/SEI 7 (2016), and vary from 0.8 to 1.6 for site class A through D. For the purposes of the zone of influence (ZOI) method, the default values of F_a that can be used in areas not covered by ASCE/SEI 7 are taken as the maximum tabulated values for site class A through D and are summarized in Table E.3(b).

Table E.3(a) Site Class

Site Class	Soil Definition
A	Hard rock
B	Medium hard rock
BC	Soft rock
C	Very dense sand or hard clay
CD	Dense sand or very stiff clay
D	Medium dense sand or stiff clay
DE	Loose sand or medium stiff clay
E	Very loose sand or soft clay

Table E.3(b) Values of F_a for Use in Areas Not Covered by ASCE/SEI 7

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
Default F_a	1.6	1.4	1.2	1.2	1.2
A	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0

Note: Use straight-line interpolation for intermediate values of S_S.

By combining Equations E.3d and E.3e, C_p can be written as shown in Equation E.3f:

$$C_p = 0.754S_{DS} = \frac{2}{3}\left(0.754S_sF_a\right) = 0.503S_sF_a \tag{E.3f}$$

The site class can be determined by a geotechnical engineer. Additionally, the site class and seismic design category of a structure are separate pieces of information. Although both terms are classified by a letter ranging from A to F, the seismic design category determines when seismic protection is required for buildings and nonstructural elements, while the site class represents the type of soil underneath a structure and its ability to resist or absorb seismic forces. Each of these designations should be determined independently in accordance with local regulations.

For example, for a *Z* factor of 0.4, resulting in a value of S_S of 1.8, the value of F_a from Table E.3(b) is 1.2 for the default site class and the following in Equation E.3g is true:

$$C_p = 0.503S_sF_a = 0.503(1.8)(1.2) = 1.09 \tag{E.3g}$$

As shown in Equation E.3a, the seismic design load, F_p, includes a variable, H_f, related to the height of the component attachment to the structure relative to the average roof height of the structure. The equation to determine H_f is illustrated in Equation E.3h.

$$H_f = \left(1 + 2.5\frac{z}{h}\right) \tag{E.3h}$$

The most conservative seismic design load assumes the seismic attachment is at 100 percent of the average roof height, where *z* = 1.0 and *h* = 1.0. In this case, the function becomes a constant equal to 3.5 in accordance with Equation E.3i:

$$H_f = \left(1 + 2.5\frac{1.0}{1.0}\right) = 1 + 2.5 = 3.5 \tag{E.3i}$$

If the seismic attachment is installed at 75 percent of the average roof height, where *z* = 0.75 and *h* = 1.0, the function becomes a constant equal to 2.875 in accordance with Equation E.3j:

$$H_f = \left(1 + 2.5\frac{0.75}{1.0}\right) = 1 + 1.875 = 2.875 \tag{E.3j}$$

Since 2.875 is 82.1 percent of 3.5, the seismic design load for seismic attachments installed at 75 percent of the average roof height can be multiplied by a factor of 0.821. Because it might be difficult to accurately measure the average roof height of the structure relative to the grade plane, 18.5.9.3.3 of NFPA 13 assumes a multiplier of 0.875 is used.

If the seismic attachment is installed at 50 percent of the average roof height, where *z* = 0.50 and *h* = 1.0, the function becomes a constant equal to 2.25 in accordance with Equation E.3k:

$$H_f = \left(1 + 2.5\frac{0.5}{1.0}\right) = 1 + 1.25 = 2.25 \tag{E.3k}$$

Since 2.25 is 64.3 percent of 3.5, the seismic design load for seismic attachments installed at 50 percent of the average roof height can be multiplied by a factor of 0.643. Because it might be difficult to accurately measure the average roof height of the structure relative to the grade plane, 18.5.9.3.4 of NFPA 13 assumes a multiplier of 0.750 is used.

E.4 Flexural Capacity of Piping.

The flexural capacity for different diameters and thicknesses of pipe were computed using allowable stress design (ASD). NFPA 13 has traditionally used ASD for design. While ASCE/SEI 7 generally uses the strength design (SD) approach, ASD is preferred for the design of piping systems. For example, the ASTM B31, *Standards of Pressure Piping*, series of piping codes are based on ASD. ASD was chosen for sprinkler piping design to limit the complexity of the analysis. Use of SD would require the use of the plastic modulus, *Z*, of the pipe rather than the elastic section modulus, *S*. Use of *Z* would trigger analysis of local and global buckling behavior of the pipe. SD is most appropriate when used with compact pipe sections that can develop the full limit capacity of the material, including strain hardening. Thin-wall pipes and materials without well-defined post-elastic behavior are not easily considered using SD.

Permissible stresses in the pipe for seismic loading are from 13.6.7 of ASCE/SEI 7. Assuming high- or limited-deformability pipe with threaded or grooved couplings, the permissible flexural stress under SD level demands is $0.7F_y$, where F_y is the yield stress of the material. Since seismic design in NFPA 13 is based on ASD, the SD capacity must be reduced to an ASD level.

The permissible flexural stress for ASD is determined by adjusting the SD level flexural capacity. The SD capacity is first reduced by a load factor to ASD levels, and then can be increased by the allowable stress increase for seismic loading. The use of an allowable stress increase for piping systems is typical when determining the strength of the pipe itself.

For fire sprinkler piping, the SD flexural capacity, M_{cap}, is reduced by a load factor of 0.7 to yield the ASD flexural capacity. The duration of load factor for the piping system, taken as 1.33, is then applied. Taking S as the section modulus of pipe, this yields an allowable moment capacity in the pipe, as shown in the following equation:

$$M_{cap} = 0.7(1.33)\left(0.7SF_y\right) = 0.65SF_y \quad \text{[E.4a]}$$

To populate Table 18.5.5.2(a) through Table 18.5.5.2(n), which give the maximum zone of influence loads, the largest reaction (due to branch lines and the tributary mass of the main) limited by flexure for a given pipe size and span between sway braces was computed.

For example, to determine the maximum permissible ZOI for a 4 in. (100 mm) diameter steel Schedule 10 main spanning 30 ft (9.1 m), first compute the flexural capacity of the pipe.

S = 1.76 in.^{3} (28800 mm^3)

F_y = 30,000 psi (2050 bar)

The flexural capacity of the pipe is as follows:

$$M_{cap} = \left(0.65F_y\right)S = (0.65)(30{,}000)(1.76) \quad \text{[E.4b]}$$

= 34,320 in.-lb (3900 kgn) = 2860 ft-lb (395 kgn)

For spans greater than 25 ft (7.6 m) and less than 40 ft (12 m), the branch lines are assumed to be located at $\frac{1}{3}$-points in the span. The point load P is associated with the branch line and tributary mass of the main and L is distance between sway braces. From Figure E.2.2(b), the maximum moment in the main, M_{max}, is

$M_{max} = 0.267PL$

Setting $M_{cap} = M_{max}$ and solving for P, the result is as follows:

$$\begin{aligned} &M_{cap} = \left(0.65F_y\right)S = 0.267PL \\ &P = \frac{M_{cap}}{0.267L} \\ &\frac{2860}{0.267(30)} = 357 \text{ lb} \end{aligned} \quad \text{[E.4c]}$$

If the required design force for braces was the same as the required design force for determining bending stress in the pipe itself, the maximum permissible ZOI load would be 3P = 1071 lb (485 kg). However, as noted above for Equation E.3a, the component resonance ductility factor, C_{AR}, and the component strength factor, R_{po}, are not the same for bracing and piping. Therefore, this ZOI load must be reduced by a factor of 1.65, determined as described in the next paragraph. For the example case, the maximum permissible ZOI load would therefore be 1071/1.65 = 649 lb (294 kg).

For rigid or cable braces, C_{AR} = 1.0 and R_{po} = 1.5. By contrast, for high- or limited-deformability non-ASME B31 piping with joints made by threading, bonding, compression couplings, or grooved couplings (representative of sprinkler piping), C_{AR} = 2.2 and R_{po} = 2.0. The [C_{AR}/R_{po}] factor, as shown in Equation E.3.a, is [2.2/2.0] = 1.1 for sprinkler pipe but is [1.0/1.5] = 0.667 for braces. Since $C_p = 0.754S_{DS}$ was developed to determine the design load for braces, if that same C_p is used to determine maximum allowable loads based on pipe bending the pipe would be overstressed by a factor of 1.1/0.667 = 1.65. Therefore, the values of F_{pw} calculated using $C_p = 0.754S_{DS}$ for braces must be divided by 1.65 such that they are applicable to bending stress limits for the pipe. This reduction has been applied in Table 18.5.5.2(a) through Table 18.5.5.2(n) such that the maximum load calculated for braces, F_{pw}, can still be used while keeping the pipe bending stress below the allowable.

E.5 Sample Seismic Calculation Using the ZOI Method.

To illustrate the application of the ZOI method, the approach can be applied to a sample problem based on the sample seismic bracing calculation in Figure A.18.5(b). The sample calculation yielded a total weight of 960 lb (435 kg)and used a seismic factor of 0.4.

A different seismic factor will be determined for this example. Assume the 4 in. (100 mm) Schedule 10 pipe is the main that will be braced and that distance between sway braces (span) is 20 ft (6.1 m). The installation is in a region of high seismicity, and based on the latitude and longitude of the building site, S_{DS} = 1.09 for the default site soil class.

To calculate the seismic load, use the equation in 18.5.9.3 to determine the seismic coefficient, C_p, and the horizontal force on the brace, F_{pw}. This is shown in Equation E.5.

$$F_{pw} = C_p W_p = 0.754 S_{DS} W_p = 0.754(1.09)(960) = 789 \text{ lb} \tag{E.5}$$

From Table 18.5.5.2(a), the maximum ZOI load, F_{pw}, for a 4 in. Schedule 10 pipe spanning 20 ft (6.1 m) is 991 lb (450 kg), which is larger than the calculated demand of 789 lb (358 kg). The 4 in. (100 mm) Schedule 10 pipe is adequate for the seismic load and a brace would be selected with a minimum capacity of 789 lb (358 kg).

If the sway brace was attached to the 2 in. (50 mm) Schedule 40 pipe, the ZOI demand F_{pw} of 789 lb (358 kg) would be compared to the maximum capacity for a 2 in. (50 mm) Schedule 40 pipe found in Table 18.5.5.2(c) and Table 18.5.5.2(d). For a 20 ft (6.1 m) span, this is 315 lb (143 kg), less than the demand of 789 lb (358 kg). A 2 in. (50 mm) pipe would be inadequate, and a sway brace would have to be added to reduce the ZOI demand, or the system pipe size increased.

E.6 Limitations of the ZOI Method.

The ZOI approach can be used for a variety of piping materials. There are, however, important limitations of which the designer should be aware. The first is that the appropriate component resonance ductility factor, C_{AR}, and component strength factor, R_{po}, must be used. To select the proper value, the piping systems must be classified as high-, limited-, or low-deformability. Definitions of these terms are given in Section 11.2 of ASCE/SEI 7. The second major assumption is that the flexural behavior of the pipe is not governed by local buckling of the pipe wall. For steel pipe, this can be achieved by observing the thickness to diameter limits given in the AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings*. Establishing the local buckling characteristics of pipe fabricated from other materials can require testing.

The tables for the maximum load, F_{pw}, in zone of influence are based on common configurations of mains and branch lines. There can be cases where the actual configuration of the piping system could generate higher stresses in the piping than assumed in the tables. For example, a main braced at 40 ft (12.2 m) intervals, with a single branch line in the center of the span, can have a smaller maximum load capacity, F_{pw}, than the tabulated value. Where the configuration of the mains and branch lines vary significantly from the assumed layout, the pipe stresses should be checked by engineering analysis.

E.7 Allowable Loads for Concrete Anchors.

This section provides step-by-step examples of the procedures for determining the allowable loads for concrete anchors as they are found in Table 18.5.12.2(a) through Table 18.5.12.2(j). Table 18.5.12.2(a) through Table 18.5.12.2(j) were developed using the prying factors found in Table E.7(a) and the representative strength design seismic shear and tension values for concrete anchors found in Table E.7(b).

Table E.7(a) Prying Factors for Table 18.5.12.2(a) through Table 18.5.12.2(j) Concrete Anchors

	Figure 18.5.12.1 Designated Angle Category								
***Pr* Range**	**A**	**B**	**C**	**D**	**E**	**F**	**G**	**H**	**I**
Lowest	2	1.1	0.7	1.2	1.1	1.1	1.4	0.9	0.8
Low	3.5	1.8	1.0	1.7	1.8	2.0	1.9	1.3	1.1
High	5.0	2.5	1.3	2.2	2.5	2.9	2.4	1.7	1.4
Highest	6.5	3.2	1.6	2.7	3.2	3.8	2.9	2.1	1.7

Table E.7(b) Representative Strength Design Seismic Shear and Tension Values Used for Concrete Anchors

Anchor Dia. (in.)	Min. Nominal Embedment (in.)	LRFD Tension (lb)	LRFD Shear (lb)	Anchor Dia. (in.)	Min. Effective Embedment (in.)	LRFD Tension (lb)	LRFD Shear (lb)
Wedge Anchors in 3000 psi (207 bar) Lightweight Sand Concrete on 4 $^1/_2$ in. Flute Width Metal Deck				**Metal Deck Inserts in 3000 psi (207 bar) Lightweight Sand Concrete on 4 $^1/_2$ in. Flute Width Metal Deck**			

Anchor Dia. (in.)	Min. Nominal Embedment (in.)	LRFD Tension (lb)	LRFD Shear (lb)	Anchor Dia. (in.)	Min. Effective Embedment (in.)	LRFD Tension (lb)	LRFD Shear (lb)
3/8	2.375	670	871	3/8	1.750	804	774
1/2	3.750	714	1489	1/2	1.750	804	837
5/8	3.875	936	1739	5/8	1.750	804	837
3/4	4.500	1372	1833	3/4	1.750	804	1617
Wedge Anchors in 3000 psi (207 bar) Lightweight Sand Concrete				**Wood Form Inserts in 3000 psi (207 bar) Lightweight Sand Concrete**			
3/8	2.375	739	1141	3/8	1.100	1358	1235
1/2	3.750	983	1955	1/2	1.690	1358	1811
5/8	3.875	1340	2091	5/8	1.750	1358	1811
3/4	4.500	1762	3280	3/4	1.750	1358	1811
Wedge Anchors in 3000 psi (207 bar) Normal Weight Concrete				**Wood Form Inserts in 3000 psi (207 bar) Normal Weight Concrete**			
3/8	2.375	1087	1170	3/8	1.100	1598	1235
1/2	3.750	1338	2574	1/2	1.690	1598	2130
5/8	3.875	2070	3424	5/8	1.750	1598	2130
3/4	4.500	3097	5239	3/4	1.750	1598	2130
Wedge Anchors in 4000 psi (276 bar) Normal Weight Concrete				**Wood Form Inserts in 4000 psi (276 bar) Normal Weight Concrete**			
3/8	2.375	1233	1170	3/8	1.100	1845	1235
1/2	3.750	1545	2574	1/2	1.690	1845	2249
5/8	3.875	2390	3900	5/8	1.750	1845	2460
3/4	4.500	3391	5239	3/4	1.750	1845	2460
Wedge Anchors in 6000 psi (414 bar) Normal Weight Concrete				**Wood Form Inserts in 6000 psi (414 bar) Normal Weight Concrete**			
3/8	2.375	1409	1170	3/8	1.100	2259	1235
1/2	3.750	1892	2574	1/2	1.690	2259	2249
5/8	3.875	2928	3900	5/8	1.750	2259	3013
3/4	4.500	4153	5239	3/4	1.750	2259	3013

E.7.1 Selecting a Wedge Anchor Using Table 18.5.12.2(a) through Table 18.5.12.2(e).

E.7.1.1 Procedure.

Step 1. Determine the ASD horizontal earthquake load F_{pw}.

Step 1a. Calculate the weight of the water-filled pipe within the zone of influence of the brace.

Step 1b. Find the applicable seismic coefficient C_p based on equations in 18.5.9.3.

Step 1c. Multiply the zone of influence weight by C_p to determine the ASD horizontal earthquake load F_{pw}.

Step 2. Select a concrete anchor from Table 18.5.12.2(a) through Table 18.5.12.2(e) with a maximum load capacity that is greater than the calculated horizontal earthquake load F_{pw} from Step 1.

Step 2a. Locate the table for the applicable concrete strength.

Step 2b. Find the column in the selected table for the applicable designated angle category (A thru I) and the appropriate prying factor *Pr* range.

Step 2c. Scan down the category column to find a concrete anchor diameter, embedment depth, and maximum load capacity that is greater than the calculated horizontal earthquake load F_{pw} from Step 1.

(ALTERNATIVE) Step 2. As an alternative to using the maximum load values in Table 18.5.12.2(a) through Table 18.5.12.2(e), select a concrete anchor that has been tested in accordance with ACI 355.2, *Post-Installed Mechanical Anchors in Concrete — Qualification Requirements and Commentary*, for seismic loading and that has an allowable strength, including the effects of prying, taken as 0.43 times the normal strength determined in accordance with Chapter 17 of ACI 318, *Building Code Requirements for Structural Concrete and Commentary*, as per 18.5.12.7.3.4.

E.7.1.2 Example.

Step 1. Zone of influence F_{pw}.

Step 1a. 40 ft of $2\frac{1}{2}$ in. Sch. 10 pipe plus 15% fitting allowance

40 × 5.89 lb/ft × 1.15 = 270.94 lb

Step 1b. Seismic coefficient C_p from 18.5.9.3

C_p = 0.35

Step 1c. F_{pw} = 0.35 × 270.94 = 94.8 lb

Step 2. Select a concrete anchor from Table 18.5.12.2(a) through Table 18.5.12.2(e).

Step 2a. Use the table for 4000 psi Normal Weight Concrete.

Step 2b. Fastener orientation “A” – assume the manufacturer's prying factor is 3.0 for the fitting. Use the *Pr* range of 2.1–3.5.

Step 2c. Allowable F_{pw} on $\frac{3}{8}$ in. dia. with 2.375 in. embedment = 138 lb and is greater than the calculated F_{pw} of 94.8 lb.

E.7.2 Calculation for Maximum Load Capacity of Concrete Anchors.

This example shows how the effects of prying and brace angle are calculated when using Table E.7(a).

E.7.2.1 Procedure.

Step 1. Determine the allowable seismic tension value (T_{allow}) and the allowable seismic shear value (V_{allow}) for the anchor, based on data found in the anchor manufacturer’s approved evaluation report. Note that, in this example, it is assumed the evaluation report provides the allowable tension and shear capacities. If this is not the case, the strength design anchor capacities must be determined using the procedures in Chapter 17 of ACI 318, which are then converted to ASD values by dividing by a factor of 1.4. As an alternative to calculating the allowable seismic tension value (T_{allow}) and the allowable seismic shear value (V_{allow}) for the anchor, the seismic tension and shear values that were used to calculate Figure 18.5.12.1 for anchor allowable load tables can be used.

Step 1a. Find the ASD seismic tension capacity (T_{allow}) for the anchor according to the strength of concrete, diameter of the anchor, and embedment depth of the anchor. Divide the ASD tension value by 2.0 and then multiply by 1.2.

Step 1b. Find the ASD seismic shear capacity (V_{allow}) for the anchor according to the strength of concrete, diameter of the anchor, and embedment depth of the anchor. Divide the ASD shear value by 2.0 and then multiply by 1.2.

Step 2. Calculate the applied seismic tension (T) and the applied seismic shear (V) based on the calculated horizontal earthquake load F_{pw}.

Step 2a. Calculate the designated angle category applied tension factor, including the effects of prying (*Pr*), using the following formulas:

Category A, B, and C

$$Pr = \frac{\left(\frac{C+A}{Tan\theta}\right) - D}{A} \quad \text{[E.7.2.1a]}$$

Category D, E, and F

$$Pr = \frac{(C+A) - \left(\frac{D}{Tan\theta}\right)}{A} \quad \text{[E.7.2.1b]}$$

Category G, H, and I

$$Pr = \frac{\left(\frac{D}{B}\right)}{Sin\theta} \quad \text{[E.7.2.1c]}$$

Step 2b. Calculate the ASD applied seismic tension (*T*) on the anchor, including the effects of prying, and when applied at the applicable brace angle from vertical and the designated angle category (A through I) using the following formula:

$$T = F_{pw} \times Pr \quad \text{[E.7.2.1d]}$$

Step 2c. Calculate the ASD applied seismic shear (*V*) on the anchor, when applied at the applicable brace angle from vertical and the designated angle category (A through I) using the following formulas:

Category A, B, and C

$$V = F_{pw} \quad \text{[E.7.2.1e]}$$

Category D, E, and F

$$V = \frac{F_{pw}}{Tan\theta} \quad \text{[E.7.2.1f]}$$

Category G, H, and I

$$V = \frac{F_{pw}}{Sin\theta} \quad \text{[E.7.2.1g]}$$

Step 3. Check the anchor for combined tension and shear loads using the following formula:

$$\left(\frac{T}{T_{allow}}\right) + \left(\frac{V}{V_{allow}}\right) \leq 1.2 \quad \text{[E.7.2.1h]}$$

Confirm that T/T_{allow} and $V/V_{allow} \leq 1.0$.

E.7.2.2 Example: Sample Calculation, Maximum Load Capacity of Concrete Anchors as Shown in Table 18.5.12.2(a) through Table 18.5.12.2(e).

In this example, a sample calculation is provided showing how the values in Table 18.5.12.2(a) through Table 18.5.12.2(e) were calculated.

Step 1. Determine the allowable seismic tension value (T_{allow}) and the allowable seismic shear value (V_{allow}) for a concrete anchor in Figure 18.5.12.1.

Step 1a. The Table E.7(b) strength design seismic tension value (T_{allow}) for a $\frac{1}{2}$ in. carbon steel anchor with $3\frac{3}{4}$ in. embedment depth in 4000 psi normal weight concrete is 1545 lb. Therefore, the allowable stress design seismic tension value (T_{allow}) is 1545/1.4/2.0 × 1.2 = 662 lb.

Step 1b. The Table E.7(b) strength design seismic shear value (V_{allow}) for a $\frac{1}{2}$ in. carbon steel anchor with $3\frac{3}{4}$ in. embedment is 2574 lb. Therefore, the allowable stress design seismic shear value (V_{allow}) is 2574/1.4/2.0 × 1.2 = 1103 lb.

Step 2. Use the applied seismic tension value (T) and the applied seismic shear value (V) based on an ASD horizontal earthquake load (F_{pw}) of 100 lb, a 30-degree brace angle from vertical, and designated angle category A.

Step 2a. Calculate the ASD applied seismic tension value (T) on the anchor, including the effects of prying, using the following formula and Figure E.7.2.2.

$$T = \frac{F_{pw}\left[\left(\frac{C+A}{Tan\theta}\right)-D\right]}{A} \quad \text{[E.7.2.2a]}$$

where:

T = applied service tension load, including the effect of prying

F_{pw} = horizontal earthquake load (F_{pw} = 170)

Tan = tangent of brace angle from vertical (*Tan*θ 0°= 0.5774)

A = 0.7500

B = 1.5000

C = 2.6250

$T = F_{pw} \times Pr$

$$T = \frac{\left\{F_{pw}\left[\left(\frac{2.625+0.75}{0.5774}\right)-1.0\right]\right\}}{0.75}$$

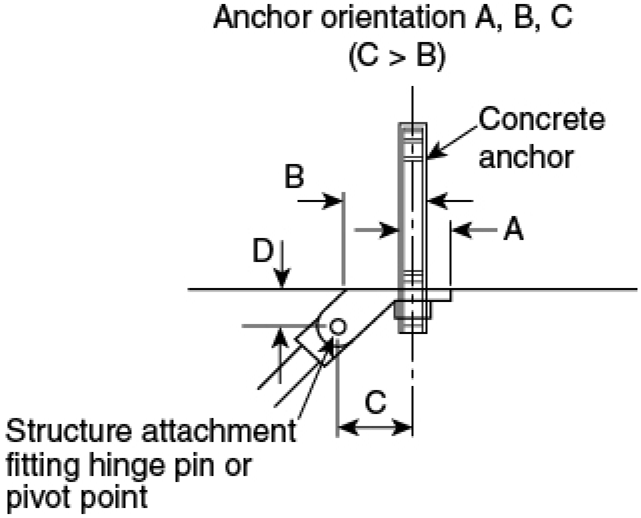
$$T = \frac{\left[F_{pw}\,(5.8452-1.0)\right]}{0.75}$$

$$T = F_{pw}\left(\frac{4.8451}{0.75}\right)$$

$T = F_{pw} \times 6.46$

T = 100 lb × 6.46 = 646 lb

Figure E.7.2.2 Concrete Anchor for Sample Calculation in E.7.2.2.



Step 2b. The ASD applied seismic shear value (V) on the anchor for anchor orientations A, B, and C is equal to the ASD horizontal earthquake load F_{pw} = 100 lb.

Step 3. Calculate the maximum allowable horizontal earthquake load F_{pw} using the formula:

$$\left(\frac{T}{T_{allow}}\right)+\left(\frac{V}{V_{allow}}\right)\leq 1.2 \quad \text{[E.7.2.2b]}$$

$$\left(\frac{646}{662}\right)+\left(\frac{100}{1103}\right)=1.0665(\leq 1.2)$$