Annex EC
(informative)

Commentary on Annex E

Acknowledgment

The development of this extensive revision to Annex E and preparation of this Commentary was funded jointly by API and the Federal Emergency Management Agency through the American Lifelines Alliance. The development of this Annex and Commentary was directed by the API Seismic Task Group with technical review by the Dynamic Analysis and Testing Committee of the Pressure Vessel Research Council.

EC.1 Scope

API 650, Annex E has been revised in it's entirety to accomplish the following:

— incorporate the newer definitions of ground motion used in the U.S. model building codes and ASCE 7;

— add a procedure to address regions outside the US where ground motions may be defined differently by local regulations;

— expand and generalize the equations to improve programming applications and reduce reliance on plots and equations where terms were combined and lacked the clarity needed to adapt to changing requirements;

— include additional requirements for hydrodynamic hoop stresses and vertical earthquake;

— include, for the convenience of the users, information and equations previously found in outside reference materials;

— revise the combination of impulsive and convective forces to use the SRSS method instead of direct sum method;

— introduce the concept of an "anchorage ratio" for clarity;

— add a foundation stability ratio requirement;

— permit the use of soil structure interaction for mechanically-anchored tanks;

— add detailing requirements for freeboard, pipe flexibility, and other components; and

— improve maintainability.

EC.2 Definitions and Notations

For additional definitions and background information, the user is referred to the following documents:

— *National Earthquake Hazard Reduction Program Provisions and Commentary*, FEMA Publications 302, 303, 368, and 369.

— ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, American Society of Civil Engineers.

— *International Building Code*, 2000 and 2003.

EC.3 Performance Basis

EC.3.1 Seismic Use Group

Tanks are classified in the appropriate Seismic Use Group based on the function and hazard to the public. Tank owner/operators may elect to specify a higher SUG as part of their risk management approach for a tank or facility. Specifying a higher SUG increases the Importance Factor, I, used to define the design acceleration parameters and indirectly influences the performance level expected of the tank. Selection of the appropriate SUG is by the owner or specifying engineer who is familiar with the risk management goals, the surrounding environment, the spill prevention, control and countermeasures plans and other factors.

SUG I is the default classification.

EC.3.1.1 Seismic Use Group III

Tanks assigned the SUG III designation are those whose function are deemed essential (i.e. critical) in nature for public safety, or those tanks that store materials that may pose a very serious risk to the public if released and lack secondary control or protection. For example, tanks serving these types of applications may be assigned SUG III unless an alternative or redundant source is available:

1) fire, rescue, and police stations;

2) hospitals and emergency treatment facilities;

3) power generating stations or other utilities required as emergency backup facilities for Seismic Use Group III facilities;

4) designated essential communication centers;

5) structures containing sufficient quantities of toxic or explosive substances deemed to be hazardous to the public but lack secondary safeguards to prevent widespread public exposure;

6) water production, distribution, or treatment facilities required to maintain water pressure for fire suppression within the municipal or public domain (not industrial).

It is unlikely that petroleum storage tanks in terminals, pipeline storage facilities and other industrial sites would be classified as SUG III unless there are extenuating circumstances.

EC.3.1.2 Seismic Use Group II

Tanks assigned the SUG II designation are those that should continue to function, after a seismic event, for public welfare, or those tanks that store materials that may pose a moderate risk to the public if released and lack secondary containment or other protection. For example, tanks serving the following types of applications may be assigned SUG II unless an alternative or redundant source is available:

1) power generating stations and other public utility facilities not included in Seismic Use Group III and required for continued operation;

2) water and wastewater treatment facilities required for primary treatment and disinfection for potable water.

EC.3.1.3 Seismic Use Group I

SUG I is the most common classification. For example, tanks serving the following types of applications may be assigned SUG I unless an alternative or redundant source is available:

1) storage tanks in a terminal or industrial area isolated from public access that has secondary spill prevention and control;

2) storage tanks without secondary spill prevention and control systems that are sufficiently removed from areas of public access such that the hazard is minimal.

EC.4 Site Ground Motion

The definition of the considered ground motion at the site is the first step in defining acceleration parameters and loads. The philosophy for defining the considered ground motion in the U.S. began changing about 1997. This new approach, which began with the evolution of the 1997 UBC and advanced through the efforts of the National Earthquake Hazard Reduction Program, was the basic resource for the new model building codes. Subsequent to the *International Building Code* 2000, ASCE 7 adopted the methods and is presently the basis for the US model building codes.

However, regulations governing seismic design for tank sites outside the U.S. may not follow this ASCE 7 approach. Therefore, this revision was written to be adaptable to these regulations. Consequently, there is no longer a definition of the “minimum” design ground motion based on US standards that applies to all sites regardless of the local regulations.

Historically, this Annex (and the U.S. standards) was based on ground motion associated with an event having a 10 % probability of exceedance in 50 years. This is an event that has a recurrence interval of 475 years. In seismically active areas where earthquakes are more frequent, such as the west coast of the US, this was a reasonable approach. In regions where earthquakes are less frequent, engineers and seismologists concluded that the hazard was under-predicted by the 475 year event. Thus, the maximum considered ground motion definition was revised to a 2 % probability of exceedance in 50 years, or a recurrence interval of about 2500 years. The economic consequences of designing to this more severe ground motion was impractical so a scaling factor was introduced based on over-strength inherently present in structures built to today’s standards. See the NEHRP Provisions for a more extensive discussion of this rationale.

The API Seismic Task Group considered setting the 475 year event as the “minimum” for application of this standard. Given the variations worldwide in defining the ground motion, it was decided that the local regulation should set the requirements. However, the owner/specifying engineer for the tank should carefully consider the risk in selecting the appropriate design motion in areas outside the U.S. The API Seismic Task Group suggests that the 475 year event be the minimum basis for defining the site ground motion for tanks.

EC.4.1 Mapped ASCE 7 Methods

The ASCE 7 maximum considered earthquake response spectrum is shown in Figure EC.1. Figure EC.2 illustrates the notations used in developing the response spectrum for the maximum considered ground motion.

EC.4.2 Site-specific Spectral Response Accelerations

In most situations, a site-specific response spectrum approach is not required. See Figure EC.3. In the rare cases that a site-specific approach is necessary, the ASCE 7 approach was adopted into the Annex. To utilize this procedure, both a probabilistic and deterministic response spectrum is developed. The site specific value is then the *lesser* of the two values.

EC.4.2.1 Site-Specific Study

<none>

EC.4.2.2 Probabilistic Site-Specific MCE Ground Motion

<none>

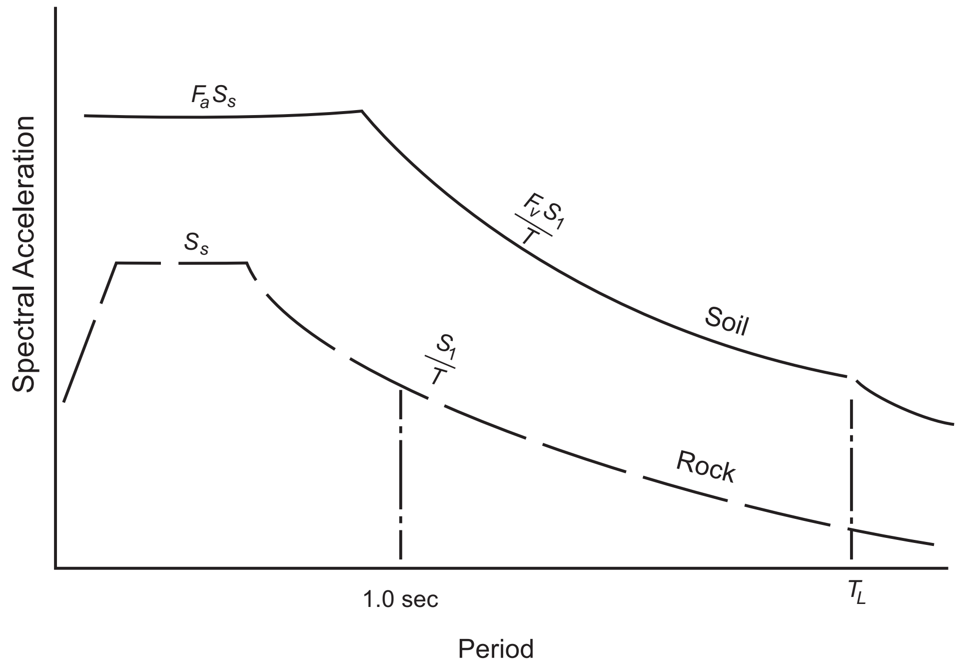


Figure EC.1—Maximum Earthquake Response Spectrum

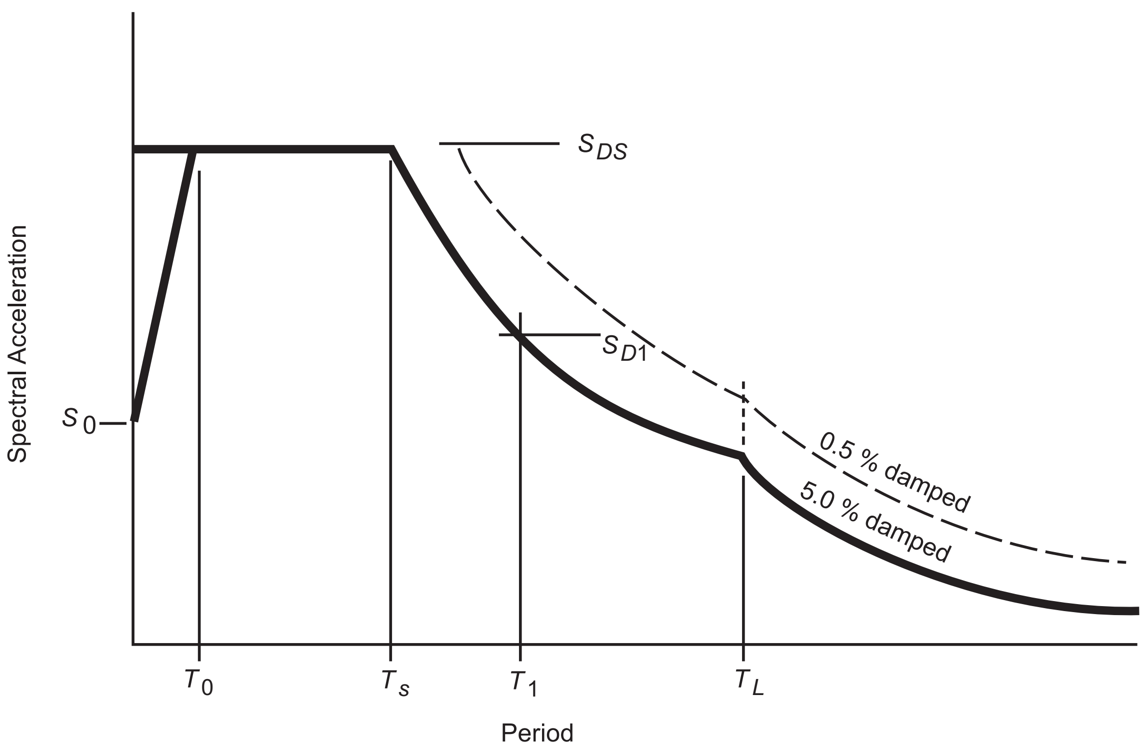


Figure EC.2—Earthquake Response Spectrum Notation

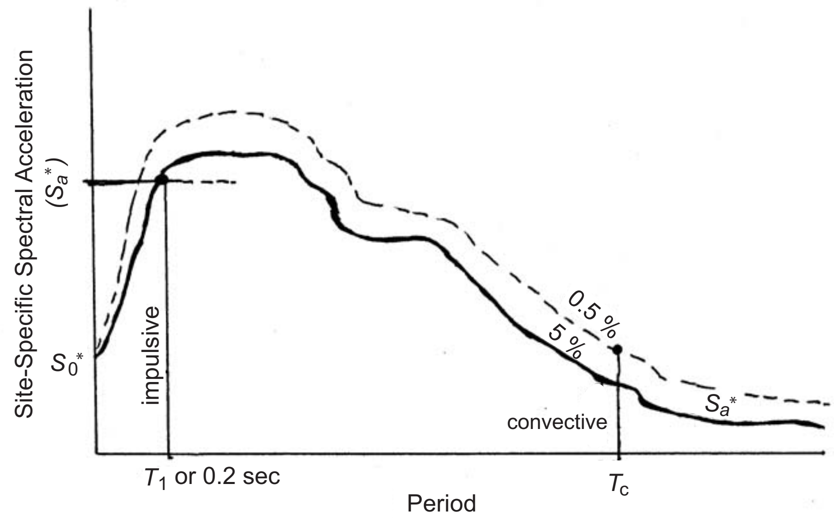


Figure EC.3—Site Specific Response Spectrum

EC.4.2.3 Deterministic Site-Specific MCE Ground Motion

In addition to the value determined for the characteristic earthquake acting on the known active faults, the deterministic values also have a lower bound limit as shown in Figure EC.4.

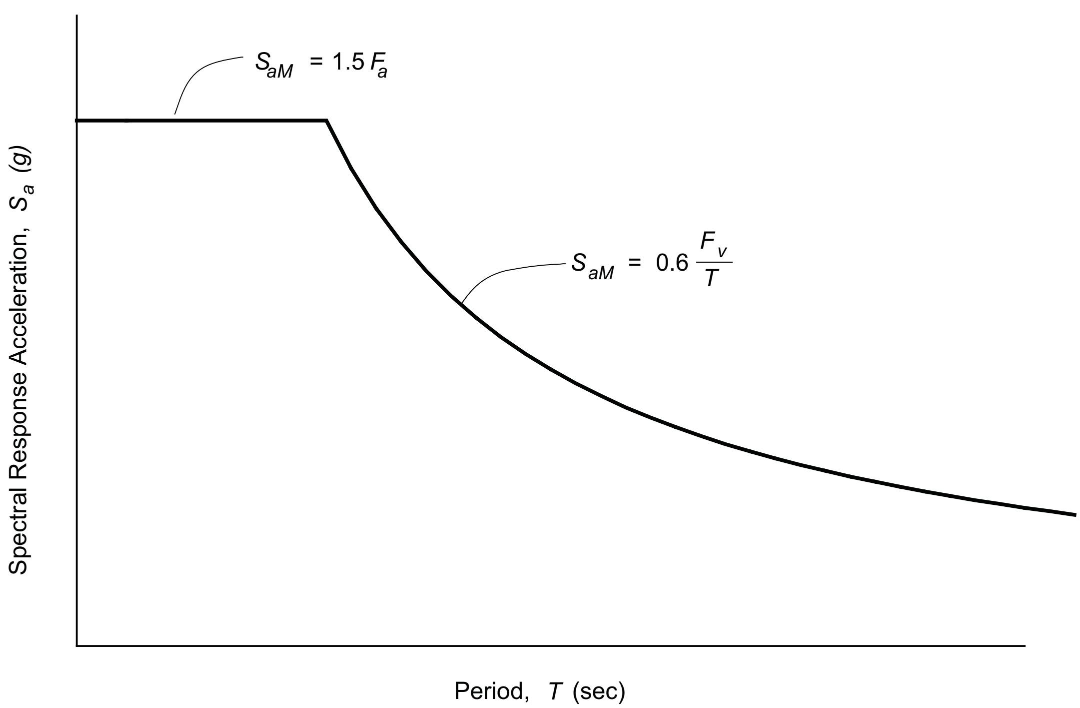


Figure EC.4—Deterministic Lower Limit on MCE Response Spectrum

EC.4.2.4 Site-Specific MCE Ground Motions

Figure EC.5 illustrates conceptually how these requirements might relate to define the site specific response spectrum.

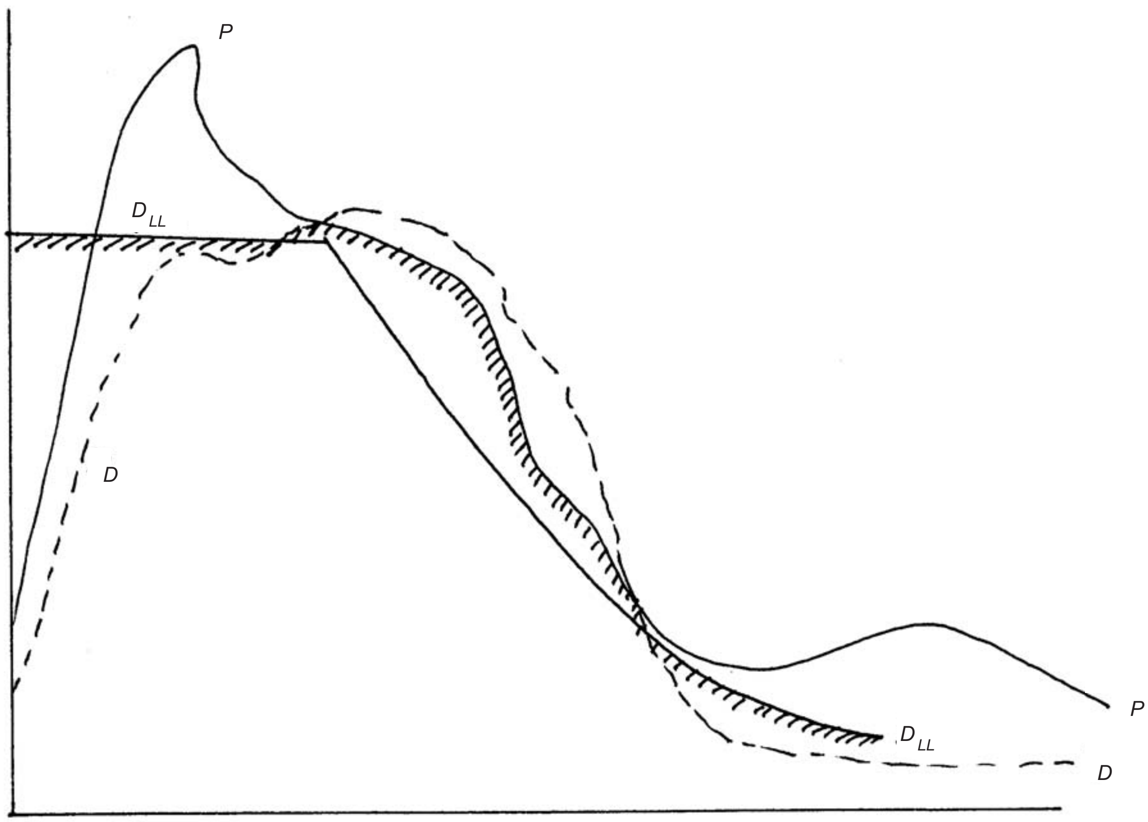


Figure EC.5—Relationship of Probabilistic and Deterministic Response Spectra

EC.4.3 Sites Not Defined By ASCE 7 Methods

The methods and equations in this Annex are best illustrated by a response spectrum curve. When the only definition of ground motion is the peak ground acceleration, the shape of the response spectrum is approximated to determine the spectral accelerations parameters. Consequently, the API Seismic Task Group recommended the relationship of S_1 and S_p defined in Equation E.4.3-2 as an approximation based on typical response spectrum curves encountered in design.

$$S_1 = 1.25S_p \tag{E.4.3-2}$$

Alternatively, if the applicable regulations have a means of determining the spectral response at the appropriate periods and damping values, those values (i.e. response spectrum) can be used, assuming that the other requirements of the Annex are met.

EC.4.4 Modifications for Site Soil Conditions

The ground motions must be amplified when the founding soils are not rock. In previous editions of the Annex, these adjustments only applied to the constant velocity and acceleration portions of the response. Since the mid-1990s, there have been dual site factors as found in ASCE 7 to define the influence of the soil on the shape and values of the ground motions. The Annex utilizes this ASCE 7 approach.

Outside the U.S., local regulations may have alternate methods of defining the influence of the soil. Such alternate methods may be used; however, if no site amplifications are defined in the local regulations, then the ASCE 7 method of addressing site amplification is required.

EC.4.5 Structural Period of Vibration

EC.4.5.1 Impulsive Natural Period

To use the methods in this Annex, the impulsive seismic acceleration parameter is independent of tank system period unless a site-specific analysis or soil structure interaction evaluation is performed. The impulsive period of the tank is nearly always less than T_S, placing it on the plateau of the response spectra. Thus, the impulsive acceleration parameter is based directly on S_{DS}. For special circumstances, a simplified procedure was included in the Annex to determine the impulsive period which was taken from the following reference:

> "Simplified Procedure for Seismic Analysis of Liquid-Storage Tanks," Malhotra, P; Wenk, T; and Wieland, M. *Structural Engineering International,* March 2000.

EC.4.5.2 Convective (Sloshing) Period

For convenience, the graphical procedure for determining the sloshing period, T_C, is included here. See Equation E.4.5.2-b and Figure EC.5.

$$T_c = K_s\sqrt{D} \quad \text{(E.4.5.2-b)}$$

where

D is the nominal tank diameter in ft;

K_S is the factor obtained from Figure EC.6 for the ratio D/H.

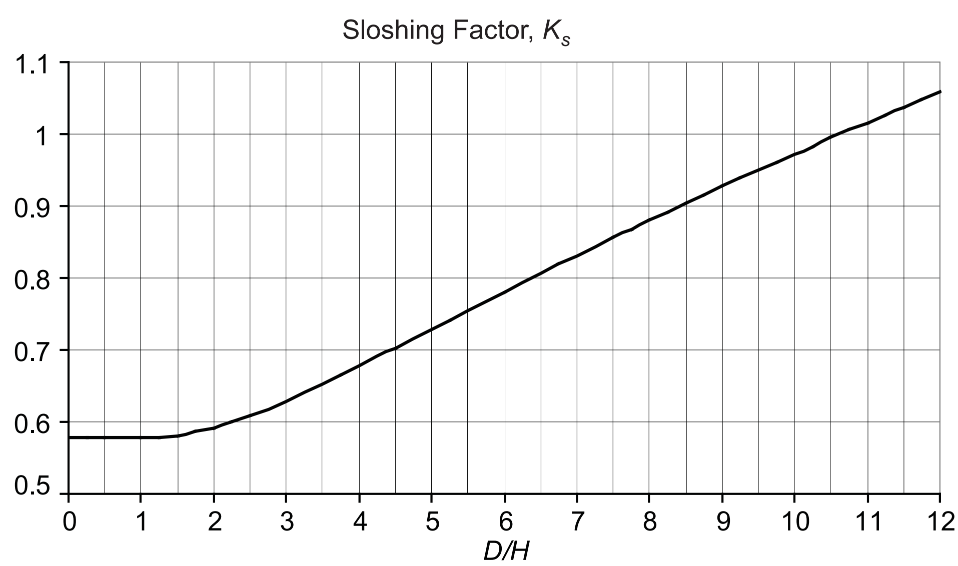


Figure EC.6—Sloshing Factor, K_s

EC.4.6 Design Spectral Response Accelerations

EC.4.6.1 Spectral Acceleration Coefficients

The acceleration parameters equations are based on the response spectrum pictured in Figure EC.7.

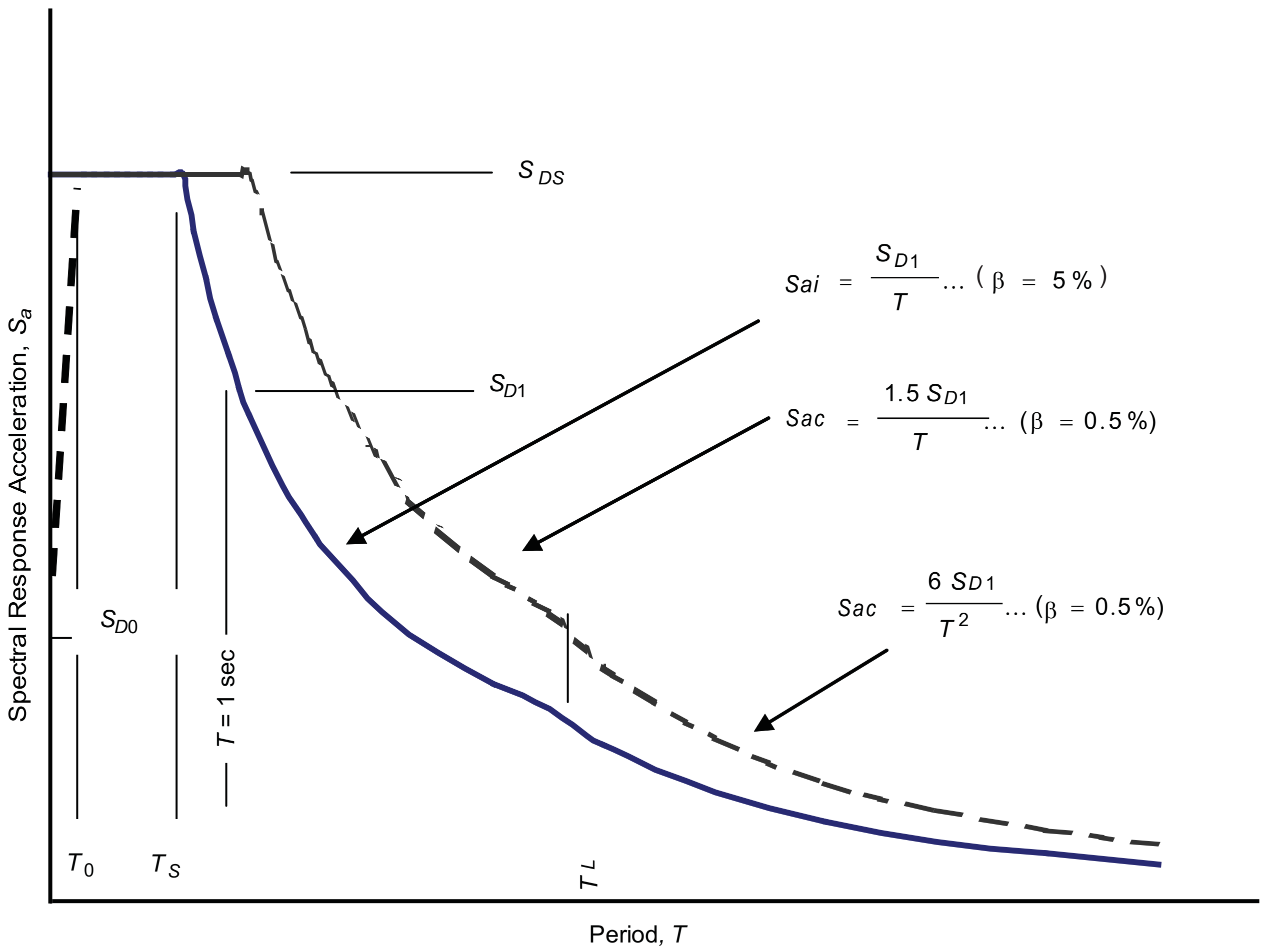


Figure EC.7—Design Response Spectra for Ground-Supported Liquid Storage Tanks

A "Q" term not included in the ASCE 7 is introduced in this Annex. "Q" is the scaling factor from the MCE, which is equal to $^2/_3$ for the ASCE 7 method. When using a recurrence interval of other than 2500 years, or another regulatory basis, "Q" should be set to the appropriate value; for most cases this is 1.0. For example, in a region outside the U.S. using the 475 year event, Q = 1.0.

For site-specific analysis, the impulsive spectral acceleration is limited to 1.5g. This is based on practical experience and observations of tank behavior. When tanks are lower profile, i.e., H/D < 0.8 and are either self- anchored or have long anchor bolt projections, the tanks can slide at the high impulsive accelerations. This sliding effectively limits the amount of force transferred into the tank. This limitation should not apply if the tank is prevented from sliding.

EC.5 Seismic Design Factors

EC.5.1 Design Forces

EC.5.1.1 Response Modification Factor

This Annex differentiates the response modification factors for impulsive and convective forces. The force reduction factor mimics the nonlinear response of the tank. There are three components to the force reduction factor R: (1) ductility R_μ, (2) damping R_β, and (3) over-strength R_Ω.

$$R = R_\mu \times R_\beta \times R_\Omega \quad \text{(EC.5.1.1-1)}$$

The ductility reduction is to account for the force reduction associated with a more flexible response. The damping reduction is to account for the force reduction associated with increased system damping. The over-strength reduction is to account for the fact that the actual strength is higher than the calculated strength.

The convective response is generally so flexible (period between 2 and 10 seconds) that any increased flexibility due to non-linearity has negligible influence on the period and damping of the convective response. It is, therefore, not justified to apply the ductility and damping reductions to the convective response—however, the over-strength reduction can still be applied. In the absence of raw data, NEHRP Technical Subcommittee 13—Non-building Structures proposed a reduction in R_w for the convective forces. After additional discussion in the ASCE Seismic Task Group, R = 1.5 (or R_{WC} of approximately 2.0) was accepted.

EC.5.1.2 Importance Factor

<none>

EC.6 Design

EC.6.1 Design Loads

Historically, steel tank standards in the US have used the direct sum of the impulsive and convective forces. Other standards do not. For example, the SRSS method of combining the impulsive and convective components is used the New Zealand Standard NZS 3106. Here is what C2.2.9.4 (Commentary) of that standard says:

"*The periods of the inertia (ed. note: impulsive) and convective responses are generally widely separated, the impulsive period being much shorter than the convective period. When responses are widely separated, near-simultaneous occurrence of peak values could occur. However, the convective response takes much longer to build up than the impulsive response, consequently the impulsive component is likely to be subsiding by the time the convective component reaches its peak. It is thus recommended that the combined impulsive and convective responses be taken as the square root of the sum of the squares of the separate components.*"

A numerical study was undertaken by the NEHRP Technical Subcommittee 13—Non-building Structures to investigate the relative accuracy of "direct sum" and SRSS methods for combining the impulsive and convective responses. In this study: (1) the impulsive period was varied between 0.05 seconds and 1 second, (2) the convective period was varied between 1 second and 20 seconds; (3) the impulsive and convective masses were assumed equal, and (4) eight different ground motions from Northridge and Landers earthquake data were used.

While, the SRSS modal combination rule does not provide the worst possible loading, it does provide the most likely loading. It has been shown that this rule is suitable for combining the impulsive and convective (sloshing) responses in tanks.

Furthermore, it should be remembered that different portions of a site response spectrum are not controlled by the same seismic event. Whereas, the short-period spectral values, which determine the impulsive response, are controlled by the closer earthquakes, the long-period spectral values, which determine the convective response, are controlled by distant, larger earthquakes. Therefore, there is already some conservatism inherent in assuming that the impulsive and convective responses will occur simultaneously.

EC.6.1.1 Effective Weight of Product

For convenience, the relationships defined in the Annex equations are graphically illustrated in Figure EC.8.

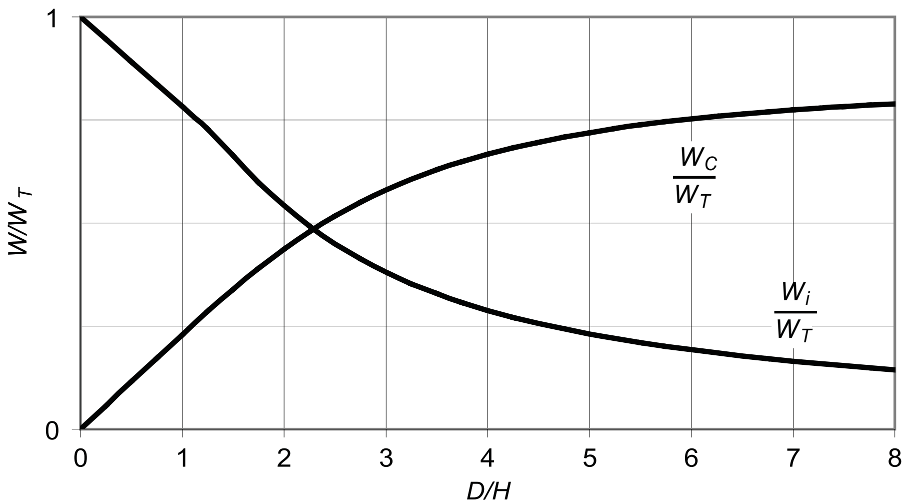


Figure EC.8—Effective Weight of Liquid Ratio

EC.6.1.2 Center of Action for Effective Forces

For convenience, the relationships defined in the Annex equations are graphically illustrated in Figure EC.9.

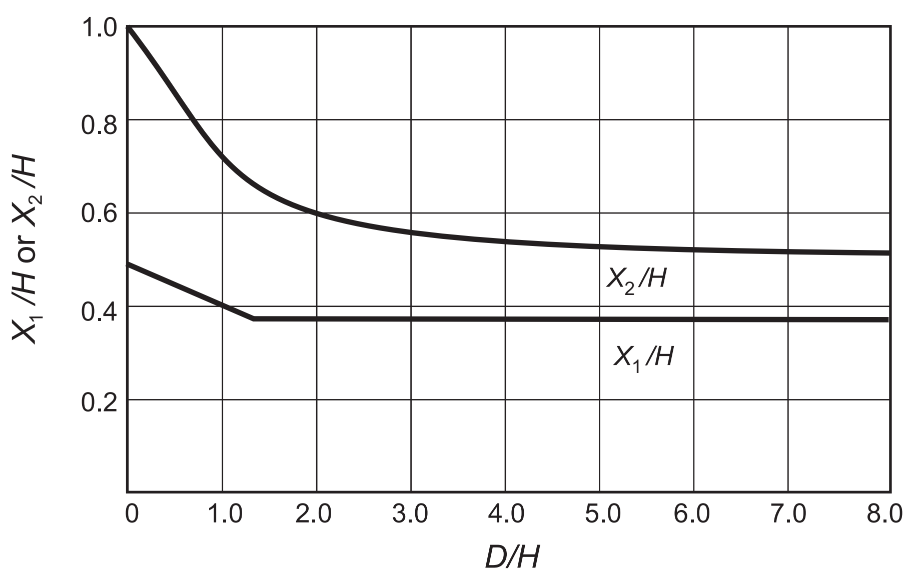


Figure EC.9—Center of Action of Effective Forces

EC.6.1.3 Vertical Seismic Effects

The vertical seismic acceleration parameter, A_v is defined by E.2.2 as A_v = (2/3) x 0.7 x S_{DS} = 0.47 S_{DS}. The 2/3 factor represents the typically applied relation between horizontal and vertical design acceleration. The 0.7 (or more precisely, 1.0 divided by 1.4) factor is the ASCE 7 adjustment for allowable stress design.

ASCE 7 sets A_v = 0.2 S_{DS}. As compared to API, this value does not include the 0.7 allowable stress factor and it does include a load combination factor of 0.3. A_v = (2/3) x 0.3 x S_{DS} = 0.2 S_{DS}. The load combination factor is not included in the API definition, because the individual load and stress equations in E.6.2.1, E.6.2.2, and E.6.2.3 include load combination effects.

Increasing A_v to 0.47 S_{DS} from the previous 0.14 S_{DS} causes the vertical seismic component of equation E.6.1.4-6 to become the largest component for determining dynamic hoop stress. This equation incorporates R_w for the impulsive and convective forces but not for vertical force. Although applying an R_w value greater than 1.0 to vertical seismic applications is not appropriate for any actions that involve buckling, applying the fully amplified elastic response of the shell hoop tension caused by the breathing response mode is also not correct. Therefore, the vertical component of the hoop stress equation is conservatively divided by a factor of 2.5. For situations where the R_{wi} is less than 2.5, the adjusting factor should be reduced to R_{wi}.

EC.6.1.4 Dynamic Liquid Hoop Forces

Calculations of hydrodynamic hoop forces were not included in previous editions of the Annex since it was not usually a governing condition for the typical petroleum storage tank. However, with larger diameter tanks, products with higher specific gravity, and vertical seismic effects, this additional check for hoop stresses was deemed to be necessary.

Increasing A_v to 0.47 S_{DS} from the previous 0.14 S_{DS} causes the vertical seismic component of equation E.6.1.4-6 to become the largest component for determining dynamic hoop stress. This equation incorporates R_w for the impulsive and convective forces but not for vertical force. Although applying an R_w value greater than 1.0 to vertical seismic applications is not appropriate for any actions that involve buckling, applying the fully amplified elastic response of the shell hoop tension caused by the breathing response mode is also not correct. Therefore, the vertical component of the hoop stress equation is conservatively divided by a factor of 2.5. For situations where the R_{wi} is less than 2.5, the adjusting factor should be reduced to R_{wi}.

EC.6.1.5 Overturning Moment

<none>

EC.6.1.6 Soil-Structure Interaction

See the NEHRP Provisions, Chapter 5 for additional information. This is applicable to mechanically anchored tanks in this Annex. The complexity and state of technology for soil structure interaction evaluations of uplifting tanks and tanks with berm foundations was considered as beyond the scope of this Annex.

EC.6.2 Resistance To Design Loads

EC.6.2.1 Anchorage

Anchorage for overturning loads may be accomplished by the inherent tank configuration and product weight (self-anchored) or by adding mechanical devices (mechanically-anchored) such as anchor bolts or straps. If a tank satisfies the requirements for self anchorage, it should not be anchored.

The methods and load combinations used to design tank anchorage have proven to be satisfactory. Alternative methods for predicting annular plate behavior and anchor bolt loads have been proposed by various researchers. The

API Seismic task Group believes that while some of these methods may more accurately depict the actual behavior of the tank, the added complexity does not significantly alter the anchorage design for the tanks usually constructed to API standards. Consequently, the simplified, but proven, method is retained.

EC.6.2.2 Maximum Longitudinal Shell Membrane Compression Stress

<none>

EC.6.2.3 Foundation

Using the calculated maximum toe pressure in the tank shell to satisfy equilibrium on self anchored flatbottom tanks produces impractical ringwall dimensions. Some yielding of soil (settlement) may occur under the shell requiring re-leveling of the tank after a seismic event. The foundations under flatbottom tanks, even tanks resting directly on earth foundations, have fared well under seismic loadings. Therefore, the seismic loading does not alter the foundation design criteria or provide justification for increased foundations for ringbearing plates.

A requirement for a mechanically-anchored tank stability check was added. This check assumes that the tank, product and foundation behave as a rigid body and is over-turning about the toe (i.e., base of the tank). This is not the actual behavior of the tank system but is a convenient model to use for checking the gross stability of the foundation. See Figure EC.10. The required factor of safety is 2.0 for this model.

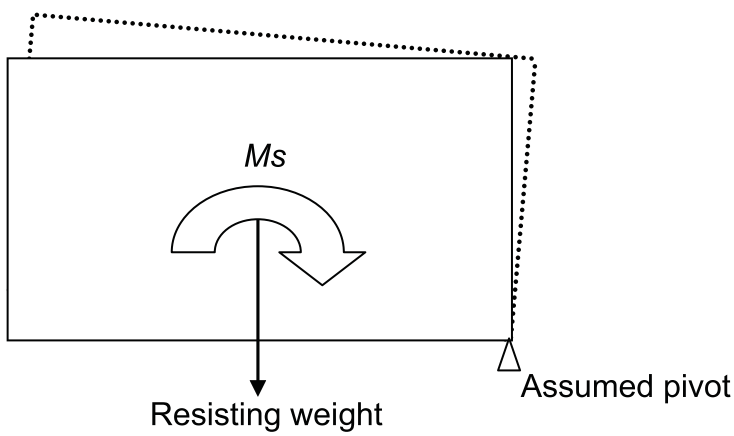


Figure EC.10—Overturning Moment

EC.6.2.4 Hoop Stress

<none>

EC.7 Detailing Requirements

EC.7.1 Shell Support

<none>

Figure EC.11 Deleted

EC.7.2 Freeboard

Freeboard is provided to reduce potential operational damage to the upper shell and roof by the impingement of the sloshing wave. In some circumstances, this damage may include tearing of the roof to shell connection and release a small amount of product. However, in almost all cases, this damage is not a structural collapse mechanism but rather an issue of operational risk and repair cost. Designing the typical API style roof and shell to resist the sloshing wave is impractical.

In the rare situation that the these provisions are applied to a tank that is completely filled and no sloshing space is provided above the maximum operating level, the entire contents of the tank should be considered an impulsive mass.

EC.7.3 Piping Flexibility

Lack of sufficient piping flexibility has been one of the leading causes of product loss observed after an earthquake. Piping designers may not recognize the movements that the tank and foundation may experience and may not provide sufficient flexibility in the piping system and supports. This overstresses the pipe and tank shell, usually causing a piping break.

Piping designers should not assume that the tank is an anchor point to resist piping loads without carefully evaluating the mechanical loads on the tank, including the compatibility of displacement. While the tank shell is relatively stiff in reacting to loads applied in the vertical direction, in most cases it is not stiff relative to the piping for radial or rotational loads.

A table of design displacements is included in the Annex. See Table E.8. These values are a compromise of practical design considerations, economics and the probability that the piping connection will be at the point of maximum uplift. If one “estimated” the tank uplift using the simplified model in the Annex, the uplift will often exceed the values in Table E.8 unless the tank is in lower ground motion regions.

Mechanically anchoring the tank to reduce piping flexibility demands should be a “last resort.” The cost of anchoring a tank that otherwise need not be anchored will often be larger than altering the piping configuration. The cost of the anchors, the foundation, and the attachment details to the shell must be weighed against piping flexibility devices or configuration changes.

Some tank designers incorporate under-bottom connections attached to the bottom out of the uplift zone. This is potentially problematic in areas where high lateral impulsive ground motion may cause the tank to slide. The tank sliding may cause a bottom failure. Properly detailed connections though the cylindrical shell are preferred.

EC.7.3.1 Method for Estimating Tank Uplift

<none>

EC.7.4 Connections

<none>

EC.7.5 Internal Components

Buckling of the roof rafters perpendicular to the primary direction of the lateral ground motion has been observed after some events. Initially, this damage was thought to be impingement damage to the rafter from the sloshing of the liquid. Presently, this buckling behavior is believed to be the result of the tendency of the flexible tank wall to oval, creating a compressive force perpendicular to the direction of the ground motion. Allowing these rafter to slip, or including an “accidental” compression load in the design of the rafter is recommended.

EC.7.6 Sliding Resistance

<none>

EC.7.7 Local Shear Transfer

<none>

EC.7.8 Connections with Adjacent Structures

<none>

DELETED

EC.8 Additional Reading

The following references are part of a large body of work addressing the behavior of tanks exposed to seismic ground motion.

[1] Hanson, R.D., *Behavior of Liquid Storage Tanks*, Report, National Academy of Sciences, Washington D.C., 1973, pp. 331 – 339.

[2] Haroun, M.A., and Housner, G.W., “Seismic Design of Liquid Storage Tanks,” *Journal of Technical Councils*, ASCE, Vol. 107, April 1981, pp. 191 – 207.

[3] Housner, G.W. 1954, *Earthquake Pressures on Fluid Containers*, California Institute of Technology.

[4] Malhotra, P.K., and Veletsos, A.S., “Uplifting Analysis of Base Plates in Cylindrical Tanks,” *Journal of Structural Division,* ASCE, Vol. 120, No. 12, 1994, pp. 3489 – 3505.

[5] Malhotra, P.K., and Veletsos, A.S., *Seismic response of unanchored and partially anchored liquid-storage tanks*, Report TR-105809. Electric Power Research Institute. Palo Alto. 1995.

[6] Malhotra, P; Wenk, T; and Wieland, M., “Simplified Procedure for Seismic Analysis of Liquid-Storage Tanks,” *Structural Engineering International,* March 2000.

[7] Manos, G. C.; Clough, R. W., *Further study of the earthquake response of a broad cylindrical liquid-storage tank model*, Report EERC 82-07, University of California, Berkeley, 1982.

[8] New Zealand Standard NZS 3106.

[9] Peek, R., and Jennings, P.C., "Simplified Analysis of Unanchored Tanks," *Journal of Earthquake Engineering and Structural Dynamics*, Vol. 16, No. 7, October 1988, pp. 1073 – 1085.

[10] Technical Information Document (TID) 7024, *Nuclear Reactors and Earthquakes*, Chap. 6 and Annex F. Published by Lockheed Aircraft Corporation under a grant from the US Dept. of Energy (formerly US Atomic Energy Commission), 1963.

[11] Veletsos, A.S., *Seismic Effects in Flexible Liquid Storage Tanks*, Proceedings of the 5th World Conference on Earthquake Engineering, Rome, Italy, Vol. 1, 1974, pp. 630 – 639.

[12] Veletsos, A.S.; Yang. J. Y., *Earthquake response of liquid storage tanks*, Proceedings of the Second Engineering Mechanics Specialty Conference. ASCE. Raleigh. 1977. pp. 1 – 24.

[13] Veletsos, A.S., "Seismic response and design of liquid storage tanks," *Guidelines for the Seismic Design of Oil and Gas Pipeline Systems*, ASCE. New York. 1984 pp. 255 – 370.

[14] Wozniak, R.S., and W.W. Mitchell. 1978, *Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks*, 1978 Proceedings—Refining Dept., Washington, D.C.: American Petroleum Institute. 57:485 – 501.

EC.9 Example Problems

1) Determining Spectral Acceleration Parameters Using ASCE 7 Method

2) Determining Spectral Acceleration Parameters Using Peak Ground Acceleration

3) Determining Spectral Acceleration Parameters Using Site-specific Response Spectrum

4) Calculating Impulsive, Convective and Combined Overturning Moment and Base Shear

5) Calculating Anchorage Ratio "J" and Self-Anchored Annular Plate

6) Calculating Hydrodynamic Hoop Stresses

7) Calculating the Overturning Stability Ratio

EC.9.1 Example Problem #1

EC.9.1.1 Determining Spectral Acceleration Parameters Using ASCE 7 Method

Required for U.S. Locations.

Seismic ground motion parameters may be determined from the ASCE 7 maps (this may be difficult in some locations due to scale); or, using digital data from USGS or IBC CD-ROM.

The results from the USGS web site for an assumed location, using the 2002 values: http://eqhazmaps.usgs.gov/index.html.

The ground motion values for the requested point:			
LOCATION	35 Lat. – 118 Long.		
DISTANCE TO			
NEAREST GRID POINT	0.00 kms		
NEAREST GRID POINT	35.00 Lat. – 118.00 Long.		
Probabilistic ground motion values, in %g, at the Nearest Grid point are:			
10 %PE in 50 yr		2 % PE in 50 yr	
PGA	23.00	38.22	<< S_o
0.2 sec SA	54.56	92.65	<< S_s
1.0 sec SA	25.35	42.09	<< S_1

Similarly, using the IBC 2000 CD-ROM *

Selecting S_s and S_1

API 650 Annex EC Example Problem	
MCE Parameters — Conterminous 48 States	
Latitude = 35.0000, Longitude = –118.0000	
Data are based on the 0.01 deg grid set	
Period SA	
(sec)	(%g)
0.2	102.7 Map Value, Soil Factor of 1.0
1.0	42.0 Map Value, Soil Factor of 1.0

Comparing to ASCE 7-02 Map, Figure 9.4.1.1(c) *

S_s = 100 % g

S_1 = 42 % g

* The ABC 2000 and ASCE 7 values are based on the USGS 1996 values. These values will be used for the example problems. The user should note that these maps are likely being revised in the later editions of these documents.

Therefore, use S_s = 103 % g, S_1 = 42 % g and S_0 = 38 % g

S_s = 103 % g

S_1 = 42 % g

S_0 = 38 % g

For this site, (from ASCE 7 maps)

T_L = 12 seconds

Assuming Site Class D, and interpolating

F_a = 1.09

(See E.4.4)

F_v = 1.58

Q = 0.67 for ASCE methods

Therefore

S_{DS} = QF_aS_s = 75 % g

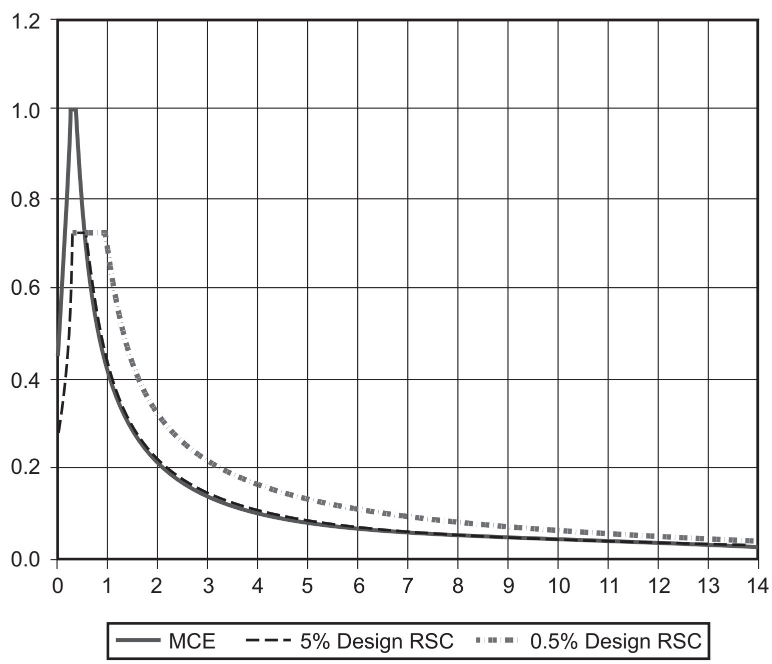
S_{D1} = QF_vS_1 = 44 % g

S_{D0} = QS_0 = 25 % g

T_s = S_{D1}/S_{DS} = 0.59 seconds

T_o = $0.2S_{D1}/S_{DS}$ = 0.12 seconds

The response spectrum can now be constructed (does not include I/R_w)



EC.9.1.2 Determine Spectral Acceleration Coefficients (See E.4.6.1)

Given:

Assume tank is self-anchored, R_w = 3.5 (see E.5.1.1)

SUG I applies, I = 1.0

Tank Diameter, D = 100 ft

Product Height, H = 40 ft

EC.9.1.3 Impulsive

$$A_i = S_{DS}\left(\frac{I}{R_{wi}}\right) = 0.75\left(\frac{1.0}{3.5}\right) = 0.21 > 0.007 \quad \text{(E.4.6.1-1)}$$

EC.9.1.4 Convective

Per E.4.5.2,

T_c = 6.09 seconds < T_L

$$A_c = KS_{D1}\left(\frac{1}{T_c}\right)\left(\frac{I}{R_{wc}}\right) = 1.5(0.44)\left(\frac{1}{6.09}\right)\left(\frac{1.0}{2}\right) = 0.054 \le .21 \quad \text{(E.4.6.1-4)}$$

EC.9.2 Example Problem #2

EC.9.2.1 Determining Spectral Acceleration Parameters Using Peak Ground Acceleration

For regions outside the U.S. where applicable.

For the same tank in Example #1, located outside the U.S.

See E.4.3.

Assuming the only parameter given is the 475 year peak ground acceleration (damping = 5 %).

This is comparable to the '*Z*' used in the earlier editions of the UBC.

Assume that regulations do not provide response spectrum.

Since 475 year recurrence interval is basis of peak ground acceleration, Q = 1.0 (no scaling).

Determine parameters:

S_p = 0.23 % g << given See Ex #1, USGS PGA for 10 % PE

S_s = 2.5 S_p = 0.58 % g

S_1 = 1.25 S_p = 0.29 % g

Assuming Site Class D, and interpolating.

No soil or site class parameters were given in the local regulations, use same as Example #1:

F_a = 1.09 (See E.4.4)

F_v = 1.58

Q = 1.00

S_0 is 475 year value

Therefore

S_{DS} = QF_aS_ss = 63 % g

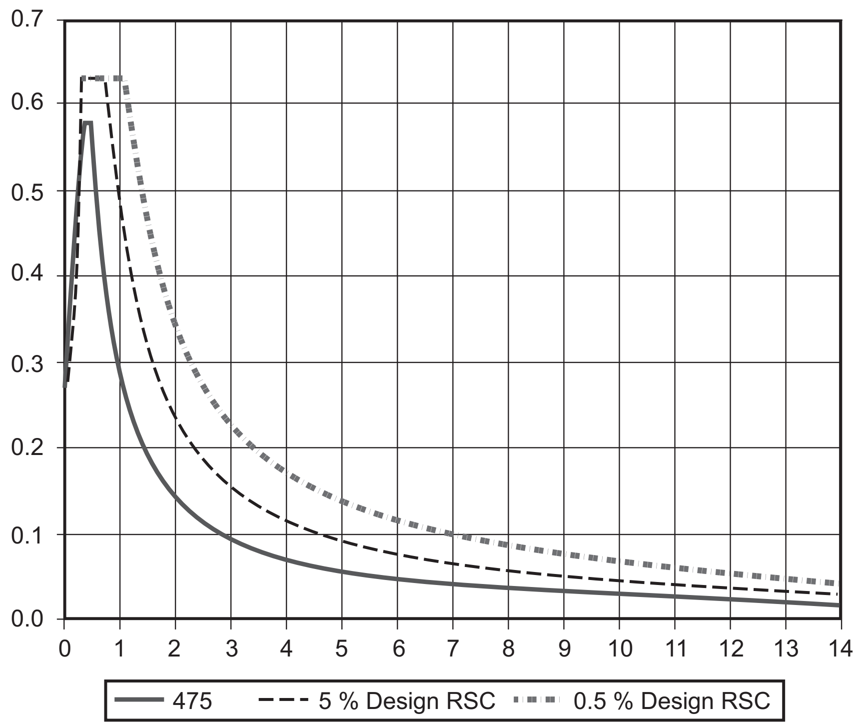
S_{D1} = QF_vS_1 = 46 % g

SD_0 = QS_0 = 23 % g

T_s = S_{D1}/S_{DS} = 0.73 seconds

T_o = $0.2S_{D1}/S_{DS}$ = 0.15 seconds

The response spectrum can now be constructed (does not include I/R_w)



The remaining calculations are similar to those shown in Example #1.

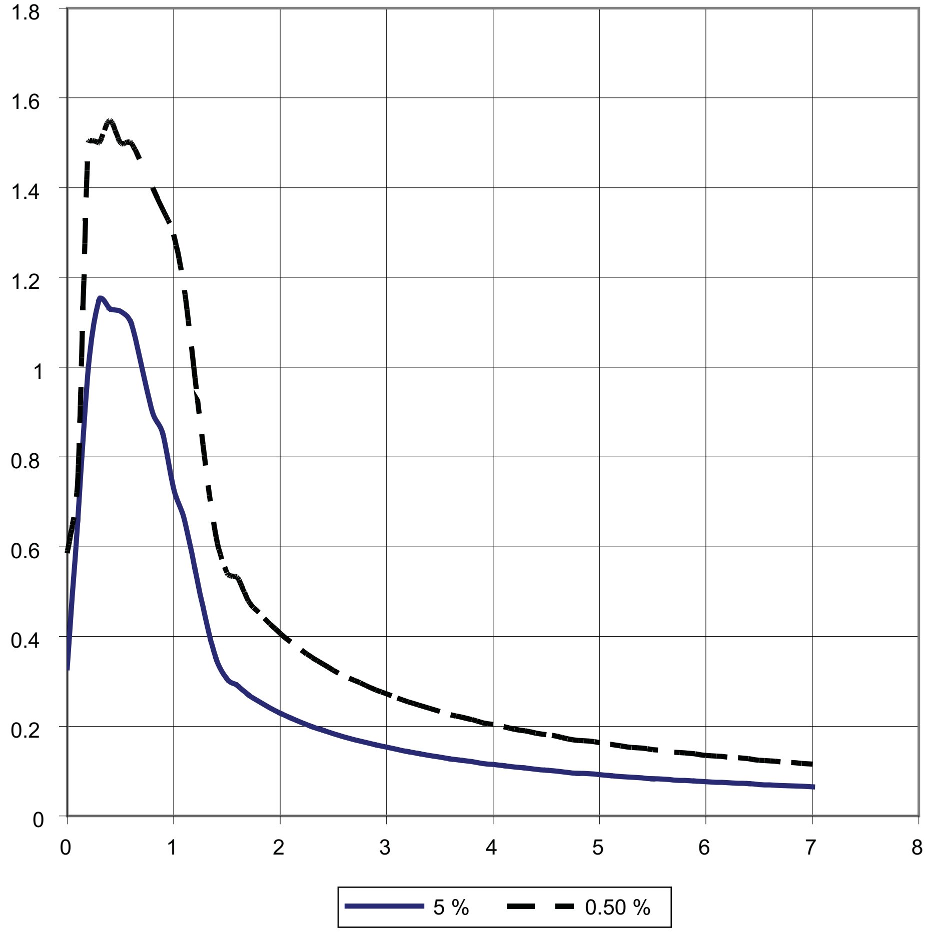
EC.9.3 Example Problem #3

EC.9.3.1 Determining Spectral Acceleration Parameters Using Site-Specific Response Spectrum

Given the following 2500 year recurrence interval site specific response spectrum.

Assume that the spectrum was developed according to the requirements of Annex E.

Also, assume that the soil/site class influences are included in the spectrum (i.e. F_a and F_v = 1.0)



From this response spectrum select the peak ground acceleration, $S_{a0}{}^*$ (the * denotes site-specific in Annex E nomenclature).

Using the 5 % curve:

$$S_{a0}{}^* = 0.33g$$

EC.9.3.2 Select the Impulsive Spectral Acceleration

There are two methods: 1) calculate the impulsive period per E.4.5.1, or Section 2) the more traditional approach—simply use the maximum value in the short period region of the curve. Using this second approach, and the 5 % spectrum:

$$S_{ai}{}^* = 1.15g$$

EC.9.3.3 Select the Convective Spectral Acceleration

Using the sloshing period form Example Problem #1, and reading from the 0.5 % curve, the convective spectral acceleration is:

$$S_{ac}{}^* = 0.13g$$

Assuming that the project specifications do not require designing for the 2500 year event, but follow Annex E:

Using Equation (E.4.6.2-1):

$$A_i = 2.5QS_{a0} * 0.550g \qquad \text{(E.4.6.2-1)}$$

Alliteratively, scale S_{ai}^* by the factor Q = 0.77g << USE

Similarly,

$A_c = QS_{ac}^*$ = 0.087g << USE

These values of A_i and A_c may be substituted into the equations in Annex E.

EC.9.4 Example Problem #4

EC.9.4.1 Calculating Impulsive, Convective and Combined Overturning Moment and Base Shear

This problem illustrates the determination of the seismic base shear and overturning forces.

Known information about the tank:

H = 40 ft

D = 100 ft

G = 0.7

W_p = 13,722,000 lb, weight of product

W_s = 213,500 lb, weight of the shell

W_r = 102,100 lb, weight of the roof (an allowance for a snow load is not required for this site)

W_f = 80,900 lb, weight of the bottom

t_s = 0.5625 in., thickness of the bottom shell course

F_y = 30,000 psi for ASTM A283, Grade C material for the bottom plate welded to the shell

S_d = 20,000 psi for ASTM A283, Grade C material for the lowest shell course

X_s = 18.0 ft (this value was assumed to be 0.45 $\times H_t$ for this sample problem)

X_r = 41.0 ft (this value was assumed to be $H_t + 1$ for this sample problem)

I = 1.00 Seismic Use Group I for a self-anchored tank

R_w = 3.5

EC.9.4.2 Problem Solution

Per E.5.1 and E.6.1.6, the equivalent lateral seismic force is given by the square root sum of the squares combination impulsive and convective forces.

The seismic base shear is determined by Equation (E.6.1-1):

$$V = \sqrt{V_i^2 + V_c^2} \qquad \text{(E.6.1-1)}$$

The seismic overturning moment at the base of the tank shell ringwall) is determined by Equation (E.6.1.5-1):

$$M_{rw} = \sqrt{[A_i(W_iX_i + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_c)]^2} \qquad \text{(E.6.1.5-1)}$$

EC.9.4.3 Determine the Impulsive Water Parameters

W_i, the impulsive weight

$D/H = 2.50 \geq 1.33$ Use Equation (E.6.1.1-1)

$$W_i = \frac{\tanh\left(0.866\frac{D}{H}\right)}{0.866\frac{D}{H}} W_p \qquad \text{(E.6.1.1-1)}$$

$= 0.450 \times 13{,}722{,}000$

$= 6{,}173{,}000$ lb

X_i, the moment arm for the impulsive product mass, see Equation (E.6.1.2.1-1)

$X_i = 0.375H = 15.0$ ft (E.6.1.2.1-1)

A_i, the impulsive spectral acceleration parameter was determined in Example Problem #1

$A_i = 0.21g$

EC.9.4.4 Determine the Convective Water Parameters

Determine W_i, the convective water weight using Equation (E.6.1.1-3)

$$W_c = 0.230\frac{D}{H}\tanh\left(\frac{3.67H}{D}\right) W_p \qquad \text{(E.6.1.1-3)}$$

$= 0.517 \times 13{,}722{,}000$

$= 7{,}095{,}000$ lb

The sloshing period was determined in Example Problem #1:

$T_c = 6.08$ seconds $< T_L = 12$ seconds

A_c was determined in Example Problem #1:

$A_c = 0.054$g

X_c, the moment arm for the convective water mass is determined by Equation (E.6.1.2.1-3):

$$X_c = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \quad \text{(E.6.1.2.1-3)}$$

$$= 0.574 \times 40$$

$$= 23.0 \text{ ft}$$

EC.9.4.5 Determine the Seismic Base Shear

The impulsive component is determined by Equation (E.6.1-2):

$$V_i = A_i(W_s + W_r + W_f + W_i) \quad \text{(E.6.1-2)}$$

$$= 0.21 \times 6{,}569{,}500$$

$$= 1{,}379{,}600 \text{ lb}$$

A_i = $0.21g$

W_s = 213,500 lb

W_r = 102,100 lb

W_f = 80,900 lb

W_i = 6,173,000 lb

The convective component is determined by Equation (E.6.1-3):

$$V_c = A_c W_c \quad \text{(E.6.1-3)}$$

$$= 0.054 \times 7{,}095{,}000$$

$$= 383{,}100 \text{ lb}$$

A_c = $0.054g$

W_c = 7,095,000 lb

The seismic base shear is:

$$V = \sqrt{V_i^2 + V_c^2}$$

$$= 1{,}431{,}800 \text{ lb}$$

EC.9.4.6 Determine the Seismic Overturning Moment

The ringwall moment is determined by Equation (E.6.1.5-1):

$$M_{rw} = \sqrt{[A_i(W_iX_i + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_c)]^2} \quad \text{(E.6.1.5-1)}$$

A_i = $0.21g$

W_i = 6,173,000 lb

X_i = 15.0 ft

W_s = 213,500 lb

X_s = 18.0 ft

W_r = 102,100 lb

X_r = 41.0 ft

= $0.21 \times 100{,}624{,}100$

= 21,131,100 ft-lb

A_c = 0.054g

W_c = 7,095,000 lb

X_c = 23.0 ft

= $0.054 \times 162{,}874{,}400$

= 8,795,200 ft-lb

The seismic overturning moment at the base of the tank shell, M_{rw}, is 22,888,400 ft-lb.

EC.9.5 Example Problem #5

EC.9.5.1 Calculating Anchorage Ratio "J" and Self-Anchored Annular Plate

Determine if the tank is suitable for the seismic overturning forces without the need for anchors.

Consideration of vertical seismic accelerations are not considered for this problem (A_v = 0).

Known information for this tank:

D = 100 ft, diameter

t = 0.5625 in., the thickness of the lowest shell course

t_a = 0.25 in., the thickness of the bottom plate welded to the shell ft

H = 40 ft

G = 0.7

S_d = 20,000 psi for ASTM A283, Grade C material for the lowest shell course

F_y = 30,000 psi for ASTM A283, Grade C material for the bottom plate welded to the shell

M_{rw} = 22,888,400 ft-lb, the seismic overturning moment at the base of the tank

W_s = 213,500 lb, the weight of the shell

W_{rs} = 61,300 lb, weight of the roof supported by the shell (assumed 60 % of W_r without snow)

w_{rs} = 195 lb/ft, the weight of the roof supported by the shell

The resisting force for a self-anchored tank is determined by Equation (E.6.2.1.1-1b):

$$w_a = 7.9 t_a \sqrt{F_y H G_e} \leq 1.28\ HDG(1 - A_v) \quad \text{(E.6.2.1.1-1b)}$$

$$= 3584 \text{ lb/ft}$$

$$w_a = 1810 \text{ lb/ft}$$

The anchorage ratio, J is:

Using Equation (E.6.2.1.1.1-2):

$$w_t = \frac{W_s}{\pi D} + w_{rs} \quad \text{(E.6.2.1.1.1-2)}$$

$$= 680 + 195$$

$$= 875 \text{ lb/ft}$$

Applying this to Equation (E.6.2.1.1.1-1):

$$J = \frac{M_{rw}}{D^2[w_t(1 - 0.4A_v) + w_a - 0.4w_{int}]} \quad \text{(E.6.2.1.1.1-1)}$$

= $0.853 < 1.54$, therefore tank is stable

For purposes of demonstration, assume M_{rw} is doubled and J is = 1.71 > 1.54, therefore tank is not stable.

With this increased load, this tank does not meet the stability requirements with a 1/4 in. thick bottom plate under the shell. Try a thickened annular plate.

Determine the required bottom thickness in order to avoid the addition of tank anchorage.

By trial-and-error, a 0.4375 in. thick annular ring will be used.

Recalculating:

t_a = 0.4375 in.

w_a = 3168 lb/ft

J = $0.566 < 1.54$, therefore tank is now stable

The minimum width of the butt welded annular ring to be provided (inside the tank) is calculated by Equation (E.6.2.1.1.2-1b):

$$L = 0.216 t_a \sqrt{F_y / HG} \quad \text{(E.6.2.1.1.2-1b)}$$

= 3.09 ft = 37.1 in.

but, L to exceed 0.035D = 3.50 ft = OK

A 0.4375 in. thickened annular plate projecting at least 37.1 in. inside the tank shell is OK providing, the check the vertical shell compression due to seismic overturning forces is met.

J = 0.566, no calculated uplift

$$\sigma_c = \left(w_t(1 + 0.4A_v) + \frac{1.273M_{rw}}{D^2}\right)\frac{1}{12t_s}$$

= 993 psi

The allowable shell compression is calculated by the following equation:

$$GHD^2/t^2 = 884,938 < 1,000,000$$

The allowable compression is given by Equation (E.6.2.2.3-2b):

$$F_C = 10^6\ t_s/(2.5D) + 600\sqrt{(GH)} \quad \text{(E.6.2.2.3-2b)}$$

$= 4925 \text{ psi} > 993 \text{ psi} = \text{OK}$

EC.9.6 Example Problem #6

EC.9.6.1 Calculating Hydrodynamic Hoop Stresses

See E.6.1.4.

Consider both lateral and vertical accelerations.

The owner has specified a vertical acceleration of 12.5 %g.

Known information about the tank:

H = 40 ft

D = 100 ft

G = 0.7

t_s = 0.5625 in., thickness of the bottom shell course

F_y = 30,000 psi for ASTM A283, Grade C material for the bottom plate welded to the shell

S_d = 20,000 psi for ASTM A283, Grade C material for the lowest shell course

E = 1.0 weld joint efficiency

A_i = 0.210 g

A_c = 0.054 g

A_v = 0.125 g

The product hydrostatic membrane hoop load at the base of the tank is:

$$N_h = 2.6(H-1)DG$$

$$= 7098 \text{ lb/in.}$$

The impulsive hoop membrane hoop force at the base of the tank is calculated by Equation (E.6.1.4-1b):

$D/H = 2.5$ $Y = H = 40$ ft

$$N_i = 4.5A_iGDH\left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right]\tanh\left(0.866\frac{D}{H}\right) \quad \text{(E.6.1.4-1b)}$$

$$= 1312 \text{ lb/in.}$$

The convective hoop membrane hoop load at the base of the tank is Equation (E.6.1.4-4b):

$D/H = 2.5$ $Y = H = 40$ ft

$$N_c = \frac{0.98A_cGD^2\cosh\left[\frac{3.68(H-Y)}{D}\right]}{\cosh\left[\frac{3.68H}{D}\right]} \quad \text{(E.6.1.4-4b)}$$

$$= 163 \text{ lb/in.}$$

The total hoop stress, including lateral and vertical seismic accelerations per Equation (E.6.1.4-b):

$$\sigma_T = \sigma_h \pm \sigma_s = \frac{N_h \pm \sqrt{N_i^2 + N_c^2 + (A_vN_h)^2}}{t} \quad \text{(E.6.1.4-b)}$$

$$= 15{,}449 \text{ psi (max)}$$

The allowable seismic hoop stress is the lesser of:

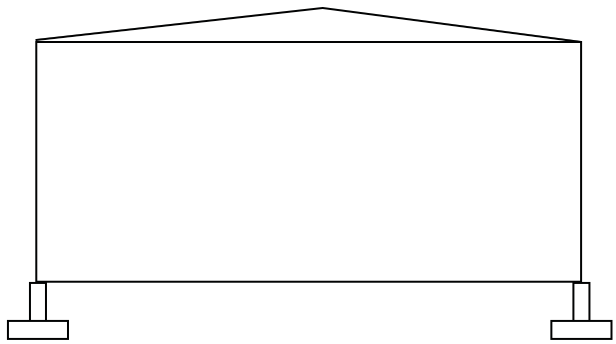
$1.333 \times S_d = 26{,}660$ psi (GOVERNS) $< 22{,}924$ psi = OK

$0.9F_y = 27{,}000$ psi

EC.9.7 Example Problem #7

EC.9.7.1 Calculating the Overturning Stability Ratio

See E.6.2.3.



See Example Problem #4:

D = 100 ft

H = 40 ft

W_p = 13,722,000 lb weight of product

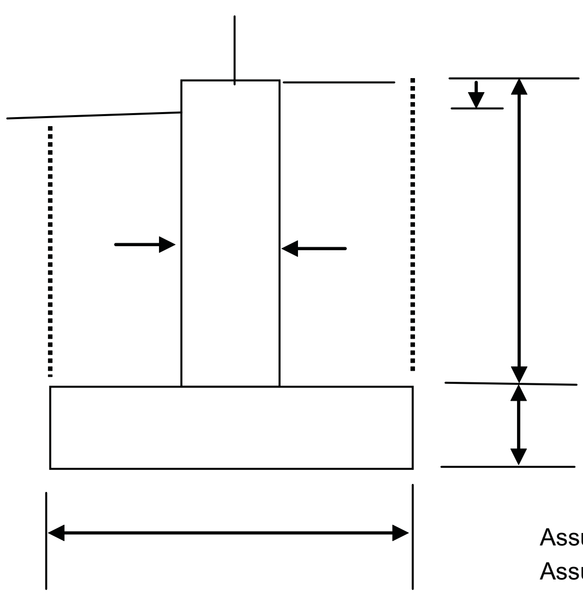
W_f = 80,900 lb weight of floor

W_T = 315,600 lb weight of tank

W_{fd} = 1,413,716 lb weight of foundation

W_g = 721,300 lb weight of soil over foundation

Assume M_s = 75,000,000 lb-ft:



Assume concrete weighs 150 lbs/cf
Assume soil weighs 100 lbs/cf

Compute weight of foundation:

$$W_{fd} = 150\pi D A_{fd} = 150\pi(100)[(2\times 6)+(3\times 6)] = 1,413,716 \text{ lb}$$

Compute weight of soil over footing.

Outside ringwall:

$$W_{go} = 100\pi(D+4\text{ ft})(2\times 5.5) = 359,400 \text{ lb}$$

$$W_{gi} = 100\pi(D-4\text{ ft})(2\times 6) = 361,900 \text{ lb}$$

Summing:

$$W_g = 721{,}300 \text{ lbs}$$

Sum moments about toe of the tank, Equation (E.6.2.3-1):

$$\frac{0.5D[W_p + W_f + W_T + W_{fd} + W_g]}{M_s} \geq 2.0 \qquad \text{(E.6.2.3-1)}$$

$$= 10.8 > 2 = \text{OK}$$