Annex E
(normative)

Seismic Design of Storage Tanks

This annex provides a number of design options requiring decisions by the Purchaser; standard requirements; recommendations; and information that supplements the basic standard. This annex becomes a requirement only when the Purchaser specifies an option covered by this annex or specifies the entire annex.

Part I—Provisions

E.1 Scope

This Annex provides minimum requirements for the design of welded steel storage tanks that may be subject to seismic ground motion. These requirements represent accepted practice for application to welded steel flat-bottom tanks supported at grade.

The fundamental performance goal for seismic design in this Annex is the protection of life and prevention of catastrophic collapse of the tank. Application of this standard does not imply that damage to the tank and related components will not occur during seismic events.

This Annex is based on the allowable stress design (ASD) methods with the specific load combinations given herein. Application of load combinations from other design documents or codes is not recommended, and may require the design methods in this Annex be modified to produce practical, realistic solutions. The methods use an equivalent lateral force analysis that applies equivalent static lateral forces to a linear mathematical model of the tank based on a rigid wall, fixed based model.

The ground motion requirements in this Annex are derived from ASCE 7, which is based on a maximum considered earthquake ground motion defined as the motion due to an event that is expected to achieve a 1-percent probability of collapse within a 50-year period. Application of these provisions as written is deemed to meet the intent and requirements of ASCE 7. Accepted techniques for applying these provisions in regions or jurisdictions where the regulatory requirements differ from ASCE 7 are also included.

The pseudo-dynamic design procedures contained in this Annex are based on response spectra analysis methods and consider two response modes of the tank and its contents—impulsive and convective. Dynamic analysis is not required nor included within the scope of this Annex. The equivalent lateral seismic force and overturning moment applied to the shell as a result of the response of the masses to lateral ground motion are determined. Provisions are included to assure stability of the tank shell with respect to overturning and to resist buckling of the tank shell as a result of longitudinal compression.

The design procedures contained in this Annex are based on a 5 % damped response spectra for the impulsive mode and 0.5 % damped spectra for the convective mode supported at grade with adjustments for site-specific soil characteristics. Application to tanks supported on a framework elevated above grade is beyond the scope of this Annex. Seismic design of floating roofs is beyond the scope of this Annex.

Optional design procedures are included for the consideration of the increased damping and increase in natural period of vibration due to soil-structure interaction for mechanically-anchored tanks.

Tanks located in regions where S_1 is less than or equal to 0.04 and S_S less than or equal to 0.15, or the peak ground acceleration for the ground motion defined by the regulatory requirements is less than or equal to 0.05g, need not be designed for seismic forces; however, in these regions, tanks in SUG III shall comply with the freeboard requirements of this Annex.

- Dynamic analysis methods incorporating fluid-structure and soil-structure interaction are permitted to be used in lieu of the procedures contained in this Annex with Purchaser approval and provided the design and construction details are as safe as otherwise provided in this Annex.

E.2 Definitions and Notations

E.2.1 Definitions

E.2.1.1
active fault
A fault for which there is an average historic slip rate of 1 mm (0.04 in.) per year or more and geologic evidence of seismic activity within Holocene times (past 11,000 years).

E.2.1.2
characteristic earthquake
An earthquake assessed for an active fault having a magnitude equal to the best-estimate of the maximum magnitude capable of occurring on the fault, but not less than the largest magnitude that has occurred historically on the fault.

E.2.1.3
risk-targeted maximum considered earthquake, MCE_R
The most severe earthquake ground motion (adjusted for targeted risk) in this annex.

E.2.1.4
site class
A classification assigned to a site based on the types of soils present and their engineering properties as defined in this Annex.

E.2.2 Notations

A Lateral acceleration coefficient, %g

A_c Convective design response spectrum acceleration parameter, %g

A_f Acceleration coefficient for sloshing wave height calculation, %g

A_i Impulsive design response spectrum acceleration coefficient, %g

A_v Vertical earthquake acceleration parameter = ($^2/_3$) x 0.7 x S_{DS} = 0.47 S_{DS}, %g

C_d Deflection amplification factor, C_d = 2.0 (self-anchored), 2.5 (mechanically anchored)

C_i Coefficient for determining impulsive period of tank system

D Nominal tank diameter, m (ft)

d_c Total thickness (100 – d_s) of cohesive soil layers in the top 30 m (100 ft)

d_i Thickness of any soil layer i (between 0 and 30 m [100 ft])

d_s Total thickness of cohesionless soil layers in the top 30 m (100 ft)

E Elastic Modulus of tank material, MPa (lbf/in.^{2})

F_a Acceleration-based site coefficient (at 0.2 sec period)

F_c Allowable longitudinal shell-membrane compression stress, MPa (lbf/in.^{2})

F_p The ratio of normal operation pressure to design pressure, with a minimum value of 0.4

F_{ty} Minimum specified yield strength of shell course, MPa (lbf/in.^{2})

F_v Velocity-based site coefficient (at 1.0 sec period)

F_y Minimum specified yield strength of bottom annulus, MPa (lbf/in.^{2})

G Design specific gravity

g Acceleration due to gravity in consistent units, m/sec^2 (ft/sec^2)

G_e Effective specific gravity including vertical seismic effects = G (1 – 0.4A_v)

H Maximum design product level, m (ft)

H_S Thickness of soil, m (ft)

I Importance factor coefficient set by seismic use group

J Anchorage ratio

K Coefficient to adjust the spectral acceleration from 5 % to 0.5 % damping = 1.5 unless otherwise specified

L Required minimum width of thickened bottom annular ring measured from the inside of the shell m (ft)

L_s Selected width of annulus (bottom or thickened annular ring) to provide the resisting force for self anchorage, measured from the inside of the shell m (ft)

t_a Thickness, excluding corrosion allowance, mm (in.) of the bottom annulus under the shell required to provide the resisting force for self anchorage. The bottom plate for this thickness shall extend radially at least the distance, L, from the inside of the shell. This term applies for self-anchored tanks only.

M_{rw} Ringwall moment—Portion of the total overturning moment that acts at the base of the tank shell perimeter, Nm (ft-lb)

M_s Slab moment (used for slab and pile cap design), Nm (ft-lb)

N Standard penetration resistance, ASTM D1586

N Average field standard penetration test for the top 30 m (100 ft)

N_c Convective hoop membrane force in tank shell, N/mm (lbf/in.)

N_{ch} Average standard penetration of cohesionless soil layers for the top 30 m (100 ft)

N_h Product hydrostatic membrane force, N/mm (lbf/in.)

N_i Impulsive hoop membrane force in tank shell, N/mm (lbf/in.)

Pf Overturning bearing force based on the maximum longitudinal shell compression at the base of shell, N/m (lbf/ft)

PI Plasticity index, ASTM D4318

Q Scaling factor from the MCE_R to the design level spectral accelerations; equals $^2/_3$ for ASCE 7

R Force reduction coefficient for strength level design methods

R_{wc} Force reduction coefficient for the convective mode using allowable stress design methods

R_{wi} Force reduction factor for the impulsive mode using allowable stress design methods

S_0 MCE_R, 5 % damped, spectral response acceleration parameter at a period of zero seconds (peak ground acceleration for a rigid structure), %g

S_1 MCE_R, 5 % damped, spectral response acceleration parameter at a period of one second, %g

S_a The 5 % damped, design spectral response acceleration parameter at any period based on mapped, probabilistic procedures, %g

$S_a{}^*$ The 5 % damped, design spectral response acceleration parameter at any period based on site-specific procedures, %g

$S_{a0}{}^*$ The 5 % damped, design spectral response acceleration parameter at zero period based on site-specific procedures, %g

S_{D1} The design, 5 % damped, spectral response acceleration parameter at one second based on the ASCE 7 methods, equals $Q\,F_v\,S_1$, %g

S_{DS} The design, 5% damped, spectral response acceleration parameter at short periods (T = 0.2 seconds) based on ASCE 7 methods, equals $Q\,F_a\,S_s$, %g

S_P Design level peak ground acceleration parameter for sites not addressed by ASCE methods. [See EC Example Problem 2 when using “Z” factor from earlier editions of API 650 and UBC. Since 475 year recurrence interval is basis of this peak ground acceleration, Q = 1.0 (no scaling).]

S_S MCE_R, 5% damped, spectral response acceleration parameter at short periods (0.2 sec), %g

s_u Undrained shear strength, ASTM D2166 or ASTM D2850

s_u Average undrained shear strength in top 30 m (100 ft)

t Thickness of the shell ring under consideration less corrosion allowance, mm (in.)

t_a Thickness, excluding corrosion allowance, mm (in.) of the bottom annulus under the shell required to provide the resisting force for self anchorage. The bottom plate for this thickness shall extend radially at least the distance, L, from the inside of the shell. this term applies for self-anchored tanks only.

t_b Thickness of tank bottom, mm (in.)

t_s Thickness of bottom shell course less corrosion allowance, mm (in.)

t_u Equivalent uniform thickness of tank shell, mm (in.)

T Natural period of vibration of the tank and contents, seconds

T_C Natural period of the convective (sloshing) mode of behavior of the liquid, seconds

T_i Natural period of vibration for impulsive mode of behavior, seconds

T_L Regional-dependent transition period for longer period ground motion, seconds

T_0 $0.2\ F_v S_1 / F_a S_S$

T_S $F_v S_1 / F_a S_S$

V Total design base shear, N (lbf)

V_c Design base shear due to the convective component of the effective sloshing weight, N (lbf)

v_s Average shear wave velocity at large strain levels for the soils beneath the foundation, m/s (ft/s)

v_s Average shear wave velocity in top one 30 m (100 ft), m/s (ft/s)

V_i Design base shear due to impulsive component from effective weight of tank and contents, N (lbf)

w Moisture content (in %), ASTM D2216

w_a Force resisting uplift in annular region, N/m (lbf/ft)

W_c Effective convective (sloshing) portion of the liquid weight, N (lbf)

W_{eff} Effective weight contributing to seismic response

W_f Weight of the tank bottom, N (lbf)

W_{fd} Total weight of tank foundation, N (lbf)

W_g Weight of soil directly over tank foundation footing, N (lbf)

W_i Effective impulsive portion of the liquid weight, N (lbf)

w_{int} Calculated design uplift load due to design pressure per unit circumferential length, N/m (lbf/ft)

W_p Total weight of the tank contents based on the design specific gravity of the product, N (lbf)

W_r Total weight of fixed tank roof including framing, any permanent attachments and 10 % of the roof balanced design snow load, S_b, N (lbf)

W_{rs} Roof load acting on the tank shell including 10 % of the roof balanced design snow load, S_b, N (lbf)

w_{rs} Roof load acting on the shell, including 10 % of the roof balanced design snow load, S_b, N/m (lbf/ft)

W_s Total weight of tank shell and appurtenances, N (lbf)

W_T Total weight of tank shell, roof, framing, product, bottom, attachments, appurtenances, and 10 % of the balanced snow load, S_b, N (lbf)

w_t Tank and roof weight acting at base of shell, N/m (lbf/ft)

X_c Height from the bottom of the tank shell to the center of action of lateral seismic force related to the convective liquid force for ringwall moment, m (ft)

X_{cs} Height from the bottom of the tank shell to the center of action of lateral seismic force related to the convective liquid force for the slab moment, m (ft)

X_i Height from the bottom of the tank shell to the center of action of the lateral seismic force related to the impulsive liquid force for ringwall moment, m (ft)

X_{is} Height from the bottom of the tank shell to the center of action of the lateral seismic force related to the impulsive liquid force for the slab moment, m (ft)

X_r Height from the bottom of the tank shell to the roof and roof appurtenances center of gravity, m (ft)

X_s Height from the bottom of the tank shell to the shell's center of gravity, m (ft)

Y Distance from liquid surface to analysis point, (positive down), m (ft)

y_u Estimated uplift displacement for self-anchored tank, mm (in.)

σ_c Maximum longitudinal shell compression stress, MPa (lbf/in.^{2})

σ_h Product hydrostatic hoop stress in the shell, MPa (lbf/in.^{2})

σ_s Hoop stress in the shell due to impulsive and convective forces of the stored liquid, MPa (lbf/in.^{2})

σ_T Total combined hoop stress in the shell, MPa (lbf/in.^{2})

μ Friction coefficient for tank sliding

ρ Density of fluid, kg/m^3 (lb/ft^3)

E.3 Performance Basis

E.3.1 Seismic Use Group

• The Seismic Use Group (SUG) for the tank shall be specified by the Purchaser. If it is not specified, the SUG shall be assigned to be SUG I.

E.3.1.1 Seismic Use Group III

SUG III tanks are those providing necessary service to facilities that are essential for post-earthquake recovery and essential to the life and health of the public; or, tanks containing substantial quantities of hazardous substances that do not have adequate control to prevent public exposure.

E.3.1.2 Seismic Use Group II

SUG II tanks are those storing material that may pose a substantial public hazard and lack secondary controls to prevent public exposure, or those tanks providing direct service to major facilities.

E.3.1.3 Seismic Use Group I

SUG I tanks are those not assigned to SUGs III or II.

E.3.1.4 Multiple Use

Tanks serving multiple use facilities shall be assigned the classification of the use having the highest SUG.

E.4 Site Ground Motion

Spectral lateral accelerations to be used for design may be based on either "mapped" seismic parameters (zones or contours), "site-specific" procedures, or probabilistic methods as defined by the design response spectra method contained in this Annex. A method for regions outside the USA where ASCE 7 methods for defining the ground motion may not be applicable is also included.

A methodology for defining the design spectrum is given in the following sections.

E.4.1 Mapped ASCE 7 Method

- For sites located in the USA, or where the ASCE 7 method is the regulatory requirement, the risk-adjusted maximum considered earthquake ground motion shall be defined as the motion due to an event that is expected to achieve a 1 % probability of collapse within a 50-year period. The following definitions apply.

— S_S is the mapped MCE_R, 5 % damped, spectral response acceleration parameter at short periods (0.2 seconds).

— S_1 is the mapped MCE_R, 5 % damped, spectral response acceleration parameter at a period of 1 second.

— S_0 is the MCE_R, 5 % damped, spectral response acceleration parameter at zero seconds (usually referred to as the peak ground acceleration). Unless otherwise specified or determined, S_0 shall be defined as $0.4S_S$ when using the mapped methods. The PGA values in ASCE 7 based on MCE_G shall not be used for S_0.

E.4.2 Site-Specific Spectral Response Accelerations

The design method for a site-specific spectral response is based on the provisions of ASCE 7. Design using site-specific ground motions should be considered where any of the following apply.

— The tank is located within 10 km (6 miles) of a known active fault.

— The structure is designed using base isolation or energy dissipation systems, which is beyond the scope of this Annex.

— The performance requirements desired by the owner or regulatory body exceed the goal of this Annex.

Site-specific determination of the ground motion is required when the tank is located on Site Class F type soils.

- If design for an MCE_R site-specific ground motion is desired, or required, the site-specific study and response spectrum shall be provided by the Purchaser as defined this section.

However, in no case shall the ordinates of the site-specific MCE response spectrum defined be less than 80 % of the ordinates of the mapped MCE_R response spectra defined in this Annex.

E.4.2.1 Site-Specific Study

A site-specific study shall account for the regional tectonic setting, geology, and seismicity. This includes the expected recurrence rates and maximum magnitudes of earthquakes on known faults and source zones, the characteristics of ground motion attenuation, near source effects, if any, on ground motions, and the effects of subsurface site conditions on ground motions. The study shall incorporate current scientific interpretations, including uncertainties, for models and parameter values for seismic sources and ground motions.

If there are known active faults identified, the maximum considered seismic spectral response acceleration at any period, S_a^*, shall be determined using both probabilistic and deterministic methods.

E.4.2.2 Probabilistic Site-Specific MCE$_R$ Ground Motion

The probabilistic site-specific MCE$_R$ ground motion shall be taken as that motion represented by a 5 % damped acceleration response spectrum that is expected to achieve a 1 % probability of collapse within a 50-year period.

E.4.2.3 Deterministic Site-Specific MCE$_R$ Ground Motion

The deterministic site-specific MCE$_R$ spectral response acceleration at each period shall be taken as an 84th percentile 5 % damped spectral response acceleration computed at that period for characteristic earthquakes individually acting on all known active faults within the region.

However, the ordinates of the deterministic site-specific MCE$_R$ ground motion response spectrum shall not be taken lower than the corresponding ordinates of the response spectrum where the value of S_S is equal to 1.5F_a and the value of S_1 is equal to 0.6F_v / T.

• E.4.2.4 Site-Specific MCE$_R$ Ground Motions

The 5 % damped site-specific MCE$_R$ spectral response acceleration at any period, S_a^*, shall be defined as the lesser of the probabilistic MCE$_R$ ground motion spectral response accelerations determined in E.4.2.2 and the deterministic MCE ground motion spectral response accelerations defined in E.4.2.3.

The response spectrum values for 0.5 % damping for the convective behavior shall be 1.5 times the 5 % spectral values unless otherwise specified by the Purchaser.

The values for sites classified as F may not be less than 80 % of the values for a Site Class E site.

E.4.3 Sites Not Defined by ASCE 7 Methods

In regions outside the USA, where the regulatory requirements for determining design ground motion differ from the ASCE 7 methods prescribed in this Annex, the following methods may be utilized.

1) A response spectrum complying with the regulatory requirements may be used providing it is based on, or adjusted to, a basis of 5 % and 0.5 % damping as required in this Annex. The values of the design spectral acceleration coefficients, A_i and A_c, which include the effects of site amplification, importance factor and response modification may be determined directly. A_i shall be based on the calculated impulsive period of the tank (see E.4.5.1) using the 5 % damped spectra, or the period may be assumed to be 0.2 seconds. A_c shall be based on the calculated convective period (see E.4.5.2) using the 0.5 % spectra.

2) If no response spectra shape is prescribed and only the peak ground acceleration, S_P, is defined, then the following substitutions shall apply:

$$S_S = 2.5\ S_P \tag{E.4.3-1}$$

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

E.4.4 Modifications for Site Soil Conditions

The MCE$_R$ spectral response accelerations for peak ground acceleration, shall be modified by the appropriate site coefficients, F_a and F_v from Table E.1 and Table E.2.

- Where the soil properties are not known in sufficient detail to determine the site class, Site Class D shall be assumed unless the authority having jurisdiction determines that Site Class E or F should apply at the site.

Table E.1—Value of F_a as a Function of Site Class

Site Class	Mapped MCE_R Spectral Response Accelerations at Short Periods				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	[a]	[a]	[a]	[a]	[a]

[a] Site-specific geotechnical investigation and dynamic site response analysis is required.

Table E.2—Value of F_v as a Function of Site Class

Site Class	Mapped MCE_R Spectral Response Accelerations at 1 Sec Periods				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	[a]	[a]	[a]	[a]	[a]

[a] Site-specific geotechnical investigation and dynamic site response analysis is required.

SITE CLASS DEFINITIONS

The Site Classes are defined as follows:

A Hard rock with measured shear wave velocity, $\bar{v}_s$ > 1500 m/s (5000 ft/sec)

B Rock with 760 m/s < $\bar{v}_s \le$ 1500 m/s (2500 ft/sec < $\bar{v}_s \le$ 5000 ft/sec)

C Very dense soil and soft rock with 360 m/s < $\bar{v}_s \le$ 760 m/s (1200 ft/sec < $\bar{v}_s \le$ 2500 ft/sec) or with either N > 50 or $\bar{s}_u$ > 100 kPa (2000 psf)

D Stiff soil with 180 m/s $\le \bar{v}_s \le$ 360 m/s (600 ft/sec $\le \bar{v}_s \le$ 1200 ft/sec) or with either 15 $\le N \le$ 50 or 50 kPa $\le \bar{s}_u \le$ 100 kPa (1000 psf $\le \bar{s}_u \le$ 2000 psf)

E A soil profile with $\bar{v}_s$ < 180 m/s (600 ft/sec) or with either N < 15, $\bar{s}_u$ < 50 kPa (1000 psf), or any profile with more than 3 m (10 ft) of soft clay defined as soil with PI > 20, $w \ge$ 40%, and $\bar{s}_u$ < 25 kPa (500 psf)

F Soils requiring site-specific evaluations:

1) Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. However, since tanks typically have an impulsive period of 0.5 secs or less, site-specific evaluations are not required but recommended to determine spectral accelerations for liquefiable soils. The Site Class may be determined as noted below, assuming liquefaction does not occur, and the corresponding values of F_a and F_v determined from Table E.1 and Table E.2.

2) Peats and/or highly organic clays (H_S > 3 m [10 ft] of peat and/or highly organic clay, where H = thickness of soil).

3) Very high plasticity clays (H_S > 8 m [25 ft] with *PI* > 75).

4) Very thick, soft/medium stiff clays (H_S > 36 m [120 ft])

The parameters used to define the Site Class are based on the upper 30 m (100 ft) of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 30 m (100 ft). The symbol i then refers to any one of the layers between 1 and n.

where

v_{si} is the shear wave velocity in m/s (ft/sec);

d_i is the *thickness* of any layer (between 0 and 30 m [100 ft]).

$$\bar{v} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad \text{(E.4.4-1)}$$

where

$\sum_{i=1}^{n} d_i$ = 30 m (100 ft);

N_i is the Standard Penetration Resistance determined in accordance with ASTM D1586, as directly measured in the field without corrections, and shall not be taken greater than 100 blows/ft.

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \tag{E.4.4-2}$$

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \tag{E.4.4-3}$$

where $\sum_{i=1}^{m} d_i = d_s$

Use only d_i and N_i for cohesionless soils.

- d_s is the total thickness of cohesionless soil layers in the top 30 m (100 ft);
- s_{ui} is the undrained shear strength in kPa (psf), determined in accordance with ASTM D2166 or D2850, and shall not be taken greater than 240 kPa (5,000 psf).

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \tag{E.4.4-4}$$

where $\sum_{i=1}^{k} d_i = d_c$

- d_c is the total thickness (100 – d_s) of cohesive soil layers in the top 30 m (100 ft);
- PI is the plasticity index, determined in accordance with ASTM D4318;
- W is the moisture content in %, determined in accordance with ASTM D2216.

STEPS FOR CLASSIFYING A SITE:

Step 1: Check for the four categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.

Step 2: Check for the existence of a total thickness of soft clay > 3 m (10 ft) where a soft clay layer is defined by: $\bar{s}_u$ < 25 kPa (500 psf) $w \geq$ 40 %, and PI > 20. If these criteria are satisfied, classify the site as Site Class E.

Step 3: Categorize the site using one of the following three methods with $\bar{v}_s$, N, and $\bar{s}_u$ computed in all cases see Table E.3:

a) $\bar{v}_s$ for the top 30 m (100 ft) ($\bar{v}_s$ method).

b) N for the top 30 m (100 ft) (N method).

c) N for cohesionless soil layers (PI < 20) in the top 30 m (100 ft) and average $\bar{s}_u$ for cohesive soil layers (PI > 20) in the top 30 m (100 ft) ($\bar{s}_u$ method).

Table E.3—Site Classification

Site Class	$\bar{v}_s$	N or N_{ch}	s_u
E	(< 180 m/s) (< 600 fps)	< 15	< 50 kPa (< 1000 psf)
D	180 m/s to 360 m/s (600 to 1200 fps)	15 to 50	50 kPa to 100 kPa (1000 psf to 2000 psf)
C	360 m/s to 760 m/s (1200 fps to 2500 fps)	> 50	100 kPa (> 2000 psf)
B	760 m/s to 1500 m/s (2500 fps to 5000 fps)		
A	> 1500 m/s (5000 fps)		

[a] If the $\bar{s}_u$ method is used and the N_{ch} and s_u criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

Assignment of Site Class B shall be based on the shear wave velocity for rock. For competent rock with moderate fracturing and weathering, estimation of this shear wave velocity shall be permitted. For more highly fractured and weathered rock, the shear wave velocity shall be directly measured or the site shall be assigned to Site Class C.

Assignment of Site Class A shall be supported by either shear wave velocity measurements on site or shear wave velocity measurements on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 30 m (100 ft), surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$.

Site Classes A and B shall not be used where there is more than 3 m (10 ft) of soil between the rock surface and the bottom of the tank foundation.

E.4.5 Structural Period of Vibration

The pseudo-dynamic modal analysis method utilized in this Annex is based on the natural period of the structure and contents as defined in this section.

E.4.5.1 Impulsive Natural Period

The design methods in this Annex are independent of impulsive period of the tank. However, the impulsive period of the tank system may be estimated by Equation E.4.5.11. See Figure E.1.

In SI units:

$$T_i = \left(\frac{1}{\sqrt{2000}}\right)\left(\frac{C_i H}{\sqrt{\frac{t_u}{D}}}\right)\left(\frac{\sqrt{\rho}}{\sqrt{E}}\right) \qquad \text{(E.4.5.1-1a)}$$

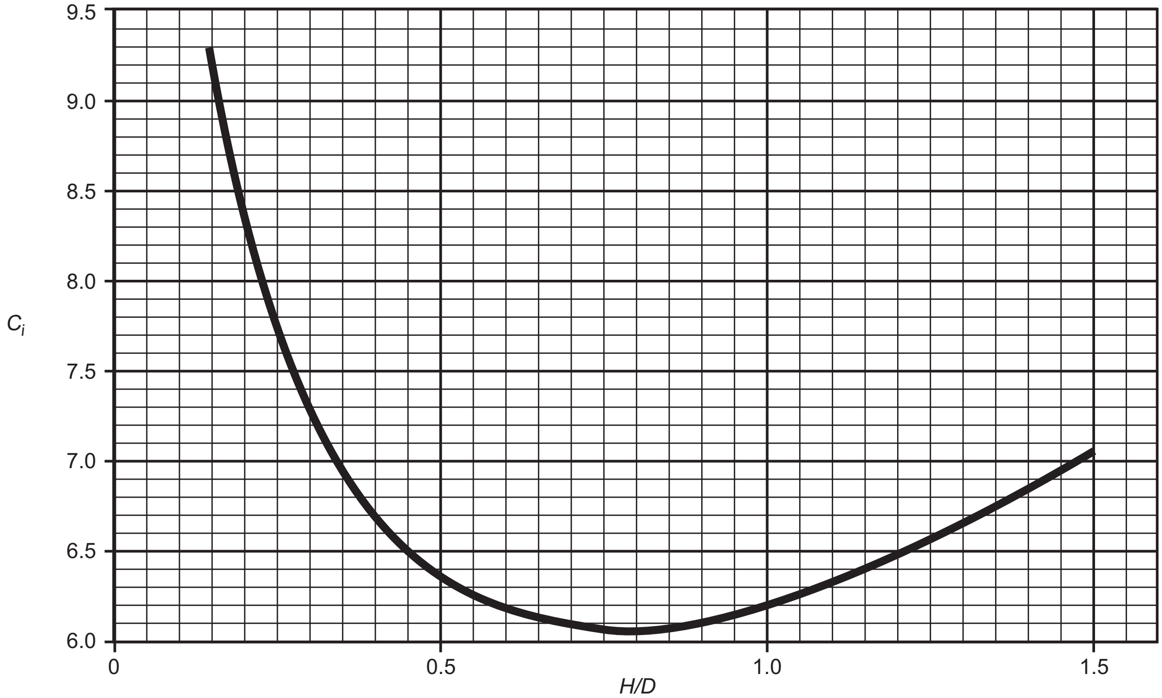

Figure E.1—Coefficient C_i

In USC units:

$$T_i = \left(\frac{1}{27.8}\right)\left(\frac{C_i H}{\sqrt{\frac{t_u}{D}}}\right)\left(\frac{\sqrt{\rho}}{\sqrt{E}}\right) \quad \text{(E.4.5.1-1b)}$$

E.4.5.2 Convective (Sloshing) Period

The first mode sloshing wave period, in seconds, shall be calculated by Equation E.4.5.2 where K_s is the sloshing period coefficient defined in Equation E.4.5.2-c:

In SI units:

$$T_c = 1.8 K_s \sqrt{D} \quad \text{(E.4.5.2-a)}$$

or, in USC units:

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

$$K_s = \frac{0.578}{\sqrt{\tanh\left(\frac{3.68H}{D}\right)}} \quad \text{(E.4.5.2-c)}$$

E.4.6 Design Spectral Response Accelerations

The design response spectrum for ground supported, flat-bottom tanks is defined by the following parameters.

• E.4.6.1 Spectral Acceleration Coefficients

When probabilistic or mapped design methods are utilized, the spectral acceleration parameters for the design response spectrum are given in Equations E.4.6.1-1 through E.4.6.1-5. Unless otherwise specified by the Purchaser, T_L shall be taken as the mapped value found in ASCE 7. For tanks falling in SUG I or SUG II, the mapped value of T_L shall be used to determine convective forces except that a value of T_L equal to 4 seconds shall be permitted to be used to determine the sloshing wave height. For tanks falling in SUG III, the mapped value of T_L shall be used to determine both convective forces and sloshing wave height except that the importance factor, I, shall be set equal to 1.0 in the determination of sloshing wave height. In regions outside the USA, where the regulatory requirements for determining design ground motion differ from the ASCE 7 methods prescribed in this Annex, T_L shall be taken as 4 seconds.

For sites where only the peak ground acceleration is defined, substitute S_P for S_0 in Equations E.4.6.1-1 through E.4.6.2-1. The scaling factor, Q, is defined as $^2/_3$ for the ASCE 7 methods. Q may be taken equal to 1.0 unless otherwise defined in the regulatory requirements where ASCE 7 does not apply. Soil amplification coefficients, F_a and F_v; the value of the importance factor, I; and the ASD response modification factors, R_{wi} and R_{wc}, shall be as defined by the local regulatory requirements. If these values are not defined by the regulations, the values in this Annex shall be used.

Impulsive spectral acceleration parameter, A_i:

$$A_i = S_{DS}\left(\frac{I}{R_{wi}}\right) = 2.5QF_aS_0\left(\frac{I}{R_{wi}}\right) \tag{E.4.6.1-1}$$

However, $A_i \geq 0.007$ (E.4.6.1-2)

and, for $S_1 \geq 0.6$:

$$A_i \geq 0.5S_1\left(\frac{I}{R_{wi}}\right) = 0.625S_P\left(\frac{I}{R_{wi}}\right) \tag{E.4.6.1-3}$$

Convective spectral acceleration parameter, A_c:

When, $T_C \leq T_L$

$$A_c = KS_{D1}\left(\frac{1}{T_c}\right)\left(\frac{I}{R_{wc}}\right) = 2.5KQF_aS_0\left(\frac{T_s}{T_c}\right)\left(\frac{I}{R_{wc}}\right) \leq A_i \tag{E.4.6.1-4}$$

When, $T_C > T_L$

$$A_c = KS_{D1}\left(\frac{T_L}{T_c^2}\right)\left(\frac{I}{R_{wc}}\right) = 2.5KQF_aS_0\left(\frac{T_sT_L}{T_c^2}\right)\left(\frac{I}{R_{wc}}\right) \leq A_i \tag{E.4.6.1-5}$$

E.4.6.2 Site-Specific Response Spectra

When site-specific design methods are specified, the seismic parameters shall be defined by Equations E.4.6.2-1 through E.4.6.2-3.

Impulsive spectral acceleration parameter:

$$A_i = 2.5\,Q\left(\frac{I}{R_{wi}}\right)S_{a0}^* \tag{E.4.6.2-1}$$

Alternatively, A_i, may be determined using either (1) the impulsive period of the tank system, or (2) assuming the impulsive period = 0.2 sec;

$$A_i = Q\left(\frac{I}{R_{wi}}\right)S_a* \qquad \text{(E.4.6.2-2)}$$

where, S_a^* is the ordinate of the 5 % damped, site-specific MCE_R response spectra at the calculated impulsive period including site soil effects. See E.4.5.1.

Exception:

- Unless otherwise specified by the Purchaser, the value of the impulsive spectral acceleration, S_a*, for flat-bottom tanks with $H/D \leq 0.8$ need not exceed 150 %g when the tanks are:

— self-anchored, or

— mechanically-anchored tanks that are equipped with traditional anchor bolt and chairs at least 450 mm (18 in.) high and are not otherwise prevented from sliding laterally at least 25 mm (1 in.).

Convective spectral acceleration:

$$A_c = QK\left(\frac{I}{R_{wc}}\right)S_a* < A_i \qquad \text{(E.4.6.2-3)}$$

where, S_a^* is the ordinate of the 5 % damped, site-specific MCE_R response spectra at the calculated convective period including site soil effects (see E.4.5.2).

Alternatively, the ordinate of a site-specific spectrum based on the procedures of E.4.2 for 0.5 % damping may be used to determine the value S_a^* with K set equal to 1.0.

E.5 Seismic Design Factors

E.5.1 Design Forces

The equivalent lateral seismic design force shall be determined by the general relationship:

$$F = AW_{\text{eff}} \qquad \text{(E.5.1-1)}$$

where

A is the lateral acceleration coefficient, %g;

W_{eff} is the effective weight.

E.5.1.1 Response Modification Factor

The response modification factor for ground supported, liquid storage tanks designed and detailed to these provisions shall be less than or equal to the values shown in Table E.4.

Table E.4—Response Modification Factors for ASD Methods

Anchorage system	R_{wi}, (impulsive)	R_{wc}, (convective)
Self-anchored	3.5	2
Mechanically-anchored	4	2

E.5.1.2 Importance Factor

- The importance factor (I) is defined by the SUG and shall be specified by the Purchaser. See E.3 and Table E.5.

Table E.5—Importance Factor (I) and Seismic Use Group Classification

Seismic Use Group	I
I	1.0
II	1.25
III	1.5

E.6 Design

E.6.1 Design Loads

Ground-supported, flat-bottom tanks, storing liquids shall be designed to resist the seismic forces calculated by considering the effective mass and dynamic liquid pressures in determining the equivalent lateral forces and lateral force distribution. This is the default method for this Annex. The equivalent lateral force base shear shall be determined as defined in the following sections.

The seismic base shear shall be defined as the square root of the sum of the squares (SRSS) combination of the impulsive and convective components unless the applicable regulations require direct sum. For the purposes of this Annex, an alternate method using the direct sum of the effects in one direction combined with 40 % of the effect in the orthogonal direction is deemed to be equivalent to the SRSS summation.

$$V = \sqrt{V_i^2 + V_c^2} \tag{E.6.1-1}$$

where

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

$$V_c = A_c W_c \tag{E.6.1-3}$$

E.6.1.1 Effective Weight of Product

The effective weights W_i and W_c shall be determined by multiplying the total product weight, W_p, by the ratios W_i/W_p and W_c/W_p, respectively, Equations E.6.1.1-1 through E.6.1.1-3.

When D/H is greater than or equal to 1.333, the effective impulsive weight is defined in Equation E.6.1.1-1:

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

When D/H is less than 1.333, the effective impulsive weight is defined in Equation E.6.1.1-2:

$$W_i = \left[1.0 - 0.218\frac{D}{H}\right] W_p \tag{E.6.1.1-2}$$

The effective convective weight is defined in Equation E.6.1.1-3:

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

E.6.1.2 Center of Action for Effective Lateral Forces

The moment arm from the base of the tank to the center of action for the equivalent lateral forces from the liquid is defined by Equations E.6.1.2.1-1 through E.6.1.2.2-3.

The center of action for the impulsive lateral forces for the tank shell, roof and appurtenances is assumed to act through the center of gravity of the component.

E.6.1.2.1 Center of Action for Ringwall Overturning Moment

The ringwall moment, M_{rw}, is the portion of the total overturning moment that acts at the base of the tank shell perimeter. This moment is used to determine loads on a ringwall foundation, the tank anchorage forces, and to check the longitudinal shell compression.

The heights from the bottom of the tank shell to the center of action of the lateral seismic forces applied to W_i and W_c, X_i and X_c, may be determined by multiplying H by the ratios X_i/H and X_c/H, respectively, obtained for the ratio D/H by using Equations E.6.1.2.1-1 through E.6.1.2.2-3.

When D/H is greater than or equal to 1.3333, the height X_i is determined by Equation E.6.1.2.1-1:

$$X_i = 0.375H \tag{E.6.1.2.1-1}$$

When D/H is less than 1.3333, the height X_i is determined by Equation E.6.1.2.1-2:

$$X_i = \left[0.5 - 0.094\frac{D}{H}\right]H \tag{E.6.1.2.1-2}$$

The height X_c is determined by Equation E.6.1.2.1-3:

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

E.6.1.2.2 Center of Action for Slab Overturning Moment

The “slab” moment, M_s, is the total overturning moment acting across the entire tank base cross-section. This overturning moment is used to design slab and pile cap foundations.

When D/H is greater than or equal to 1.333, the height X_{is} is determined by Equation E.6.1.2.2-1:

$$X_{is} = 0.375\left[1.0 + 1.333\left(\frac{0.866\frac{D}{H}}{\tanh\left(0.866\frac{D}{H}\right)} - 1.0\right)\right]H \tag{E.6.1.2.2-1}$$

When D/H is less than 1.333, the height X_{is} is determined by Equation E.6.1.2.2-2:

$$X_{is} = \left[0.500 + 0.060\frac{D}{H}\right]H \qquad \text{(E.6.1.2.2-2)}$$

The height, X_{cs}, is determined by Equation E.6.1.2.2-3:

$$X_{cs} = \left[1.0 - \frac{\cosh\left(\frac{3.67H}{D}\right) - 1.937}{\frac{3.67H}{D}\sinh\left(\frac{3.67H}{D}\right)}\right]H \qquad \text{(E.6.1.2.2-3)}$$

E.6.1.3 Vertical Seismic Effects

- When specified (see Line 8 in the Data Sheet), vertical acceleration effects shall be considered as acting in both upward and downward directions and combined with lateral acceleration effects by the SRSS method unless a direct sum combination is required by the applicable regulations. Vertical acceleration effects for hydrodynamic hoop stresses shall be combined as shown in E.6.1.4. Vertical acceleration effects need not be combined concurrently for determining loads, forces, and resistance to overturning in the tank shell except as applied in the equations of this Annex.

- The vertical seismic acceleration parameter shall be taken as 0.47S_{DS}, unless otherwise specified by the Purchaser. Alternatively, the Purchaser may specify the vertical ground motion acceleration. That acceleration shall be multiplied by 0.7 to obtain the vertical acceleration parameter, A_v. The total vertical seismic force shall be:

$$F_v = \pm A_v W_{\text{eff}} \qquad \text{(E.6.1.3-1)}$$

Vertical seismic effects shall be considered in the following when specified:

— shell hoop tensile stresses (see E.6.1.4);

— shell-membrane compression (see E.6.2.2);

— anchorage design (see E.6.2.1);

— fixed roof components (see E.7.5);

— sliding (see E.7.6);

— foundation design (see E.6.2.3).

- In regions outside the USA where the regulatory requirements differ from the methods prescribed in this Annex, the vertical acceleration parameter and combination with lateral effects may be applied as defined by the governing regulatory requirements.

E.6.1.4 Dynamic Liquid Hoop Forces

Dynamic hoop tensile stresses due to the seismic motion of the liquid shall be determined by the following formulas:

For $D/H \geq 1.33$:

In SI units:

$$N_i = 8.48A_iGDH\left[\frac{Y}{H} - 0.5\left(\frac{Y}{H}\right)^2\right]\tanh\left(0.866\frac{D}{H}\right) \quad \text{(E.6.1.4-1a)}$$

or, in USC units:

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

For $D/H < 1.33$ and $Y < 0.75D$:

In SI units:

$$N_i = 5.22A_iGD^2\left[\frac{Y}{0.75D} - 0.5\left(\frac{Y}{0.75D}\right)^2\right] \quad \text{(E.6.1.4-2a)}$$

or, in USC units:

$$N_i = 2.77A_iGD^2\left[\frac{Y}{0.75D} - 0.5\left(\frac{Y}{0.75D}\right)^2\right] \quad \text{(E.6.1.4-2b)}$$

For $D/H < 1.33$ and $Y \geq 0.75D$:

In SI units:

$$N_i = 2.6A_iGD^2 \quad \text{(E.6.1.4-3a)}$$

or, in USC units:

$$N_i = 1.39A_iGD^2 \quad \text{(E.6.1.4-3b)}$$

For all proportions of D/H:

In SI units:

$$N_c = \frac{1.85A_cGD^2\cosh\left[\frac{3.68(H-Y)}{D}\right]}{\cosh\left[\frac{3.68H}{D}\right]} \quad \text{(E.6.1.4-4a)}$$

or, in USC units:

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

When the Purchaser specifies that vertical acceleration need not be considered (i.e. $A_v = 0$), the combined hoop stress shall be defined by Equation E.6.1.4-5. The dynamic hoop tensile stress shall be directly combined with the product hydrostatic design stress in determining the total stress.

$$\sigma_T = \sigma_h \pm \sigma_s = \frac{N_h \pm \sqrt{N_i^2 + N_c^2}}{t} \quad \text{(E.6.1.4-5)}$$

When vertical acceleration is specified.

$$\sigma_T = \sigma_h \pm \sigma_s = \frac{N_h \pm \sqrt{N_i^2 + N_c^2 + (A_v N_h / 2.5)^2}}{t} \quad \text{(E.6.1.4-6)}$$

E.6.1.5 Overturning Moment

- The seismic overturning moment at the base of the tank shell shall be the SRSS summation of the impulsive and convective components multiplied by the respective moment arms to the center of action of the forces unless otherwise specified.

Ringwall Moment, M_{rw}:

$$M_{rw} = \sqrt{[A_i(W_i X_i + W_s X_s + W_r X_r)]^2 + [A_c(W_c X_c)]^2} \quad \text{(E.6.1.5-1)}$$

Slab Moment, M_s:

$$M_s = \sqrt{[A_i(W_i X_{is} + W_s X_s + W_r X_r)]^2 + [A_c(W_c X_{cs})]^2} \quad \text{(E.6.1.5-2)}$$

Unless a more rigorous determination is used, the overturning moment at the bottom of each shell ring shall be defined by linear approximation using the following.

1) If the tank is equipped with a fixed roof, the impulsive shear and overturning moment is applied at the top of the shell.

2) The impulsive shear and overturning moment for each shell course is included based on the weight and centroid of each course.

3) The overturning moment due to the liquid is approximated by a linear variation that is equal to the ringwall moment, M_{rw} at the base of the shell to zero at the maximum liquid level.

E.6.1.6 Soil-Structure Interaction

- If specified by the Purchaser, the effects of soil-structure interaction on the effective damping and period of vibration may be considered for tanks in accordance with ASCE 7 with the following limitations,

— Tanks shall be equipped with a reinforced concrete ringwall, mat or similar type foundation supported on grade. Soil-structure interaction effects for tanks supported on granular berm or pile type foundation are outside the scope of this Annex.

— The tanks shall be mechanically anchored to the foundation.

— The effective damping factor for the structure-foundation system shall not exceed 20 %.

E.6.2 Resistance to Design Loads

The allowable stress design (ASD) method is utilized in this Annex. Allowable stresses in structural elements applicable to normal operating conditions may be increased by 33 % when the effects of the design earthquake are included unless otherwise specified in this Annex.

E.6.2.1 Anchorage

Resistance to the design overturning (ringwall) moment at the base of the shell may be provided by:

— The weight of the tank shell, weight of roof reaction on shell W_{rs}, and by the weight of a portion of the tank contents adjacent to the shell for self-anchored tanks. Tanks are permitted to be designed without anchorage when they meet the requirements for self-anchored tanks listed in E.6.2.1.1.

— Mechanical anchorage devices.

E.6.2.1.1 Self-Anchored

For self-anchored tanks, a portion of the contents may be used to resist overturning. The anchorage provided is dependent on the assumed width of a bottom annulus uplifted by the overturning moment. The resisting annulus may be a portion of the tank bottom or a separate butt-welded annular ring. The overturning resisting force of the annulus that lifts off the foundation shall be determined by Equation E.6.2.1.1-1 except as noted below:

In SI units:

$$w_a = 99 t_a \sqrt{F_y H G_e} \leq 201.1\ H D G_e \quad \text{(E.6.2.1.1-1a)}$$

or, in USC units:

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

Equation E.6.2.1.1-1 for w_a applies whether or not a thickened bottom annulus is used. If w_a exceeds the limit of 201.1 HDG_e, (1.28 HDG_e) the value of L shall be set to 0.035D and the value of w_a shall be set equal to 201.1 HDG_e, (1.28 HDG_e). A value of L defined as L_s that is less than that determined by the equation found in E.6.2.1.1.2-1 may be used. If a reduced value L_s is used, a reduced value of w_a shall be used as determined below:

In SI units:

$$w_a = 5742\ H G_e L_s \quad \text{(E.6.2.1.1-2a)}$$

In USC units

$$w_a = 36.5\ H G_e L_s \quad \text{(E.6.2.1.1-2b)}$$

The tank is self-anchored providing the following conditions are met:

1) The resisting force is adequate for tank stability (i.e. the anchorage ratio, $J \leq 1.54$).

2) The maximum width of annulus for determining the resisting force is 3.5 % of the tank diameter.

3) The shell compression satisfies E.6.2.2.

4) The required annulus plate thickness does not exceed the thickness of the bottom shell course.

5) Piping flexibility requirements are satisfied.

E.6.2.1.1.1 Anchorage Ratio, *J* (see Table E.6)

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

where

$$w_t = \left[\frac{W_s}{\pi D} + w_{rs}\right] \tag{E.6.2.1.1.1-2}$$

Table E.6—Anchorage Ratio Criteria

Anchorage Ratio *J*	Criteria
$J \le 0.785$	No calculated uplift under the design seismic overturning moment. The tank is self-anchored.
$0.785 < J \le 1.54$	Tank is uplifting, but the tank is stable for the design load providing the shell compression requirements are satisfied. Tank is self-anchored.
$J > 1.54$	Tank is not stable and cannot be self-anchored for the design load. Modify the annular ring if $L < 0.035D$ is not controlling or add mechanical anchorage.

E.6.2.1.1.2 Annular Ring Requirements

The thickness of the tank bottom plate provided under the shell may be greater than or equal to the thickness of the general tank bottom plate with the following restrictions.

NOTE In thickening the bottom annulus, the intent is not to force a thickening of the lowest shell course, thereby inducing an abrupt thickness change in the shell, but rather to impose a limit on the bottom annulus thickness based on the shell design.

1) The thickness, t_a, corresponding with the final w_a in Equation E.6.2.1.1.1-1 and Equation E.6.2.1.1.1-2 shall not exceed the first shell course thickness, t_s, less the shell corrosion allowance.

2) Nor shall the thickness, t_a, used in Equation E.6.2.1.1.1-1 and Equation E.6.2.1.1.1-2 exceed the actual thickness of the plate under the shell less the corrosion allowance for tank bottom.

3) When the bottom plate under the shell is thicker than the remainder of the tank bottom, the minimum projection, L, of the supplied thicker annular ring inside the tank wall shall be the greater of 0.45 m (1.5 ft) or as determined in equation (E.6.2.1.1.2-1); however, L need not be greater than 0.035 *D*:

In SI units:

$$L = 0.01723 t_a \sqrt{F_y/(HG_e)} \tag{E.6.2.1.1.2-1a}$$

or, in USC units:

$$L = 0.216 t_a \sqrt{F_y/(HG_e)} \tag{E.6.2.1.1.2-1b}$$

E.6.2.1.1.3 Annular Ring Welding Requirements

For tanks in SUG III and located where S_{DS} = 0.5g or greater, butt-welded annular plates shall be required. Annular plates exceeding 10 mm ($^3/_8$ in.) thickness shall be butt-welded. The weld of the shell to the bottom annular plate shall be checked for the design uplift load.

E.6.2.1.2 Mechanically-Anchored

- If the tank configuration is such that the self-anchored requirements cannot be met, the tank must be anchored with mechanical devices such as anchor bolts or straps. The design of the anchorage and its attachment to the tank shall meet the minimum requirements of 5.12. In addition, hooked anchor bolts (L- or J-shaped embedded bolts) or other anchorage systems based solely on bond or mechanical friction shall not be used when anchors are required for seismic load. Post-installed anchors may be used provided that testing validates their ability to develop yield load in the anchor under cyclic loads in cracked concrete and meet the requirements of ACI 355.

E.6.2.2 Maximum Longitudinal Shell-Membrane Compression Stress

E.6.2.2.1 Shell Compression in Self-Anchored Tanks

The maximum longitudinal shell compression stress at the bottom of the shell when there is no calculated uplift, $J \leq 0.785$, shall be determined by the formula:

In SI units:

$$\sigma_c = \left(w_t(1 + 0.4A_v) + \frac{1.273M_{rw}}{D^2}\right)\frac{1}{1000t_s} \tag{E.6.2.2.1-1a}$$

or, in USC units:

$$\sigma_c = \left(w_t(1 + 0.4A_v) + \frac{1.273M_{rw}}{D^2}\right)\frac{1}{12t_s} \tag{E.6.2.2.1-1b}$$

The maximum longitudinal shell compression stress at the bottom of the shell when there is calculated uplift, $J > 0.785$, shall be determined by the formula:

In SI units:

$$\sigma_c = \left(\frac{w_t(1 + 0.4A_v) + w_a}{0.607 - 0.18667[J]^{2.3}} - w_a\right)\frac{1}{1000t_s} \tag{E.6.2.2.1-2a}$$

or, in USC units:

$$\sigma_c = \left(\frac{w_t(1 + 0.4A_v) + w_a}{0.607 - 0.18667[J]^{2.3}} - w_a\right)\frac{1}{12t_s} \tag{E.6.2.2.1-2b}$$

E.6.2.2.2 Shell Compression in Mechanically-Anchored Tanks

The maximum longitudinal shell compression stress at the bottom of the shell for mechanically-anchored tanks shall be determined by the formula:

In SI units:

$$\sigma_c = \left(w_t(1 + 0.4A_v) + \frac{1.273M_{rw}}{D^2}\right)\frac{1}{1000t_s} \quad \text{(E.6.2.2.2-1a)}$$

or, in USC units:

$$\sigma_c = \left(w_t(1 + 0.4A_v) + \frac{1.273M_{rw}}{D^2}\right)\frac{1}{12t_s} \quad \text{(E.6.2.2.2-1b)}$$

E.6.2.2.3 Allowable Longitudinal Shell-Membrane Compression Stress in Tank Shell

The maximum longitudinal shell compression stress σ_c must be less than the seismic allowable stress F_C, which is determined by the following formulas and includes the 33% increase for ASD. These formulas for F_C, consider the effect of internal pressure due to the liquid contents.

When GHD^2/ t^2 is $\geq$ 44 (SI units) (10^6 USC units),

In SI units:

$$F_C = 83\ t_s/D \quad \text{(E.6.2.2.3-1a)}$$

or, in USC units:

$$F_C = 10^6\ t_s/D \quad \text{(E.6.2.2.3-1b)}$$

In SI units:

When GHD^2/ t^2 is < 44:

$$F_C = 83t_s/(2.5D) + 7.5\sqrt{(GH)} < 0.5F_{ty} \quad \text{(E.6.2.2.3-2a)}$$

or, in USC units:

When GHD^2/t^2 is less than 1×10^6:

$$F_C = 10^6\ t_s/(2.5D) + 600\sqrt{(GH)} < 0.5F_{ty} \quad \text{(E.6.2.2.3-2b)}$$

If the thickness of the bottom shell course calculated to resist the seismic overturning moment is greater than the thickness required for hydrostatic pressure, less corrosion allowance, then the calculated thickness of each upper shell course for hydrostatic pressure shall be increased in the same proportion, unless a special analysis is made to determine the seismic overturning moment and corresponding stresses at the bottom of each upper shell course (see E.6.1.5).

E.6.2.3 Foundation

Foundations and footings for mechanically-anchored flat-bottom tanks shall be proportioned to resist peak anchor uplift and overturning bearing pressure. Product and soil load directly over the ringwall and footing may be used to resist the maximum anchor uplift on the foundation, provided the ringwall and footing are designed to carry this eccentric loading.

Product load shall not be used to reduce the anchor load.

When vertical seismic accelerations are applicable, the product load directly over the ringwall and footing.

1) When used to resist the maximum anchor uplift on the foundation, the product pressure shall be multiplied by a factor of $(1 – 0.4A_v)$ and the foundation ringwall and footing shall be designed to resist the eccentric loads with or without the vertical seismic accelerations.

2) When used to evaluate the bearing (downward) load, the product pressure over the ringwall shall be multiplied by a factor of $(1 + 0.4A_v)$ and the foundation ringwall and footing shall be designed to resist the eccentric loads with or without the vertical seismic accelerations.

The overturning stability ratio for mechanically-anchored tank system excluding vertical seismic effects shall be 2.0 or greater as defined in Equation E.6.2.3-1.

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

where

$W_T = W_s + W_r + W_p + W_f$

Ringwalls for self-anchored flat-bottom tanks shall be proportioned to resist overturning bearing pressure based on the maximum longitudinal shell compression force at the base of the shell in Equation E.6.2.3-2. Slabs and pile caps for self-anchored tanks shall be designed for the peak loads determined in E.6.2.2.1.

$$P_f = \left(w_t(1 + 0.4A_v) + \frac{1.273M_{rw}}{D^2}\right) \qquad \text{(E.6.2.3-2)}$$

E.6.2.4 Hoop Stresses

The maximum allowable hoop tension membrane stress for the combination of hydrostatic product and dynamic membrane hoop effects shall be the lesser of:

— the basic allowable membrane in this standard for the shell plate material increased by 33 %; or,

— $0.9F_y$ times the joint efficiency where F_y is the lesser of the published minimum yield strength of the shell material or weld material.

E.7 Detailing Requirements

E.7.1 Shell Support

Self-anchored tanks resting on concrete ring walls or slabs shall have a uniformly supported annulus under the shell. The foundation must be supplied to the tolerances required in 7.5.5 in order to provide the required uniform support for Item b, Item c, and Item d below. Uniform support shall be provided by one of the following methods.

a) Shimming and grouting the annulus.

b) Using fiberboard or other suitable padding.

c) Using double butt-welded bottom or annular plates resting directly on the foundation. Annular plates or bottom plates under the shell may utilize back-up bar welds if the foundation is notched to prevent the back-up bar from bearing on the foundation.

d) Using closely spaced shims (without structural grout) provided that the localized bearing loads are considered in the tank wall and foundation to prevent local crippling and spalling.

Mechanically-anchored tanks shall be shimmed and grouted.

E.7.2 Freeboard

• Sloshing of the liquid within the tank or vessel shall be considered in determining the freeboard required above the top capacity liquid level. A minimum freeboard shall be provided per Table E.7. See E.4.6.1. Purchaser shall specify whether freeboard is desired for SUG I tanks. Freeboard is required for SUG II and SUG III tanks. The height of the sloshing wave above the product design height can be estimated by:

$$\delta_s = 0.42\ DA_f \quad \text{(see Note c in Table E.7)} \tag{E.7.2-1}$$

For SUG I and II,

When $T_C \leq 4$,
$$A_f = KS_{D1}I\left(\frac{1}{T_C}\right) = 2.5KQF_aS_0I\left(\frac{T_S}{T_C}\right) \tag{E.7.2-2}$$

When $T_C > 4$,
$$A_f = KS_{D1}I\left(\frac{4}{T_C^2}\right) = 2.5KQF_aS_0I\left(\frac{4T_S}{T_C^2}\right) \tag{E.7.2-3}$$

For SUG III,

When $T_C \leq T_L$,
$$A_f = KS_{D1}I\left(\frac{1}{T_C}\right) = 2.5KQF_aS_0I\left(\frac{T_S}{T_C}\right) \tag{E.7.2-4}$$

When $T_C > T_L$,
$$A_f = KS_{D1}I\left(\frac{T_L}{T_C^2}\right) = 2.5KQF_aS_0I\left(\frac{T_ST_L}{T_C^2}\right) \tag{E.7.2-5}$$

Table E.7—Minimum Required Freeboard

Value of S_{DS}	SUG I	SUG II	SUG III
$S_{DS} < 0.33g$	(a)	(a)	δ_s (c)
$S_{DS} \geq 0.33g$	(a)	$0.7\delta_s$ (b)	δ_s (c)

a. A freeboard of $0.7\delta_s$ is recommended for economic considerations but not required.

b. A freeboard equal to $0.7\delta_s$ is required unless one of the following alternatives are provided.
1. Secondary containment is provided to control the product spill.
2. The roof and tank shell are designed to contain the sloshing liquid.

c. Freeboard equal to the calculated wave height, δ_s, is required unless one of the following alternatives are provided.
1. Secondary containment is provided to control the product spill.
2. The roof and tank shell are designed to contain the sloshing liquid.

E.7.3 Piping Flexibility

Piping systems connected to tanks shall consider the potential movement of the connection points during earthquakes and provide sufficient flexibility to avoid release of the product by failure of the piping system. The piping system and supports shall be designed so as to not impart significant mechanical loading on the attachment to the tank shell. Local loads at piping connections shall be considered in the design of the tank shell. Mechanical devices which add flexibility such as bellows, expansion joints, and other flexible apparatus may be used when they are designed for seismic loads and displacements.

Unless otherwise calculated, piping systems shall provide for the minimum displacements in Table E.8 at working stress levels (with the 33 % increase for seismic loads) in the piping, supports and tank connection. The piping system and tank connection shall also be designed to tolerate 1.4C_d times the working stress displacements given in Table E.8 without rupture, although permanent deformations and inelastic behavior in the piping supports and tank shell is permitted. For attachment points located above the support or foundation elevation, the displacements in Table E.8 shall be increased to account for drift of the tank or vessel.

Table E.8—Design Displacements for Piping Attachments

Condition	ASD Design Displacement mm (in.)
Mechanically-anchored tanks	
Upward vertical displacement relative to support or foundation:	25 (1)
Downward vertical displacement relative to support or foundation:	13 (0.5)
Range of horizontal displacement (radial and tangential) relative to support or foundation:	13 (0.5)
Self-anchored tanks	
Upward vertical displacement relative to support or foundation:	
Anchorage ratio less than or equal to 0.785:	25 (1)
Anchorage ratio greater than 0.785:	100 (4)
Downward vertical displacement relative to support or foundation:	
For tanks with a ringwall/mat foundation:	13 (0.5)
For tanks with a berm foundation:	25 (1)
Range of horizontal displacement (radial and tangential) relative to support or foundation:	50 (2)

The values given in Table E.8 do not include the influence of relative movements of the foundation and piping anchorage points due to foundation movements (such as settlement or seismic displacements). The effects of foundation movements shall be included in the design of the piping system design, including the determination of the mechanical loading on the tank or vessel consideration of the total displacement capacity of the mechanical devices intended to add flexibility.

When $S_{DS} < 0.1$, the values in Table E.8 may be reduced to 70 % of the values shown.

E.7.3.1 Method for Estimating Tank Uplift

The maximum uplift at the base of the tank shell for a self-anchored tank constructed to the criteria for annular plates (see E.6.2.1) may be approximated by Equation E.7.3.1-1. This upward vertical displacement may be used in lieu of the Table E-8 values and need not be multiplied by 1.4C_d to determine displacement for piping designs:

In SI units:

$$y_u = \frac{12.10 F_y L^2}{(t_b - CA)} \tag{E.7.3.1-1a}$$

Or, in USC units:

$$y_u = \frac{F_y L^2}{83{,}300(t_b - CA)} \quad \text{(E.7.3.1-1b)}$$

where

t_b is the calculated annular ring t hold-down

E.7.4 Connections

Connections and attachments for other lateral force resisting components shall be designed to develop the strength of the component (e.g. minimum published yield strength, F_y in direct tension, plastic bending moment), or 4 times the calculated element design load.

Penetrations, manholes, and openings in shell components shall be designed to maintain the strength and stability of the shell to carry tensile and compressive membrane shell forces.

The bottom connection on a self-anchored flat-bottom tank shall be located inside the shell a sufficient distance to minimize damage by uplift. As a minimum, the distance measured to the edge of the connection reinforcement shall be the width of the calculated self-anchored bottom hold-down plus 300 mm (12 in.)

E.7.5 Internal Components

The attachments of internal equipment and accessories which are attached to the primary liquid- or pressure-retaining shell or bottom, or provide structural support for major components shall be designed for the lateral loads due to the sloshing liquid in addition to the inertial forces.

• Seismic design of roof framing and columns shall be made if specified by the Purchaser. The Purchaser shall specify live loads and amount of vertical acceleration to be used in seismic design of the roof members. Columns shall be designed for lateral liquid inertia loads and acceleration as specified by the Purchaser. Seismic beam-column design shall be based upon the primary member allowable stresses set forth in AISC (ASD), increased by one-third for seismic loading.

Internal columns shall be guided or supported to resist lateral loads (remain stable) even if the roof components are not specified to be designed for the seismic loads, including tanks that need not be designed for seismic ground motion in this Annex (see E.1).

E.7.6 Sliding Resistance

The transfer of the total lateral shear force between the tank and the subgrade shall be considered.

For self-anchored flat-bottom steel tanks, the overall horizontal seismic shear force shall be resisted by friction between the tank bottom and the foundation or subgrade. Self-anchored storage tanks shall be proportioned such that the calculated seismic base shear, V, does not exceed V_s:

The friction coefficient, μ, shall not exceed 0.4. Lower values of the friction coefficient should be used if the interface of the bottom to supporting foundation does not justify the friction value above (e.g., leak detection membrane beneath the bottom with a lower friction factor, smooth bottoms, etc.).

$$V_s = \mu(W_s + W_r + W_f + W_p)(1.0 - 0.4A_v) \quad \text{(E.7.6-1)}$$

No additional lateral anchorage is required for mechanically-anchored steel tanks designed in accordance with this Annex even though small movements of approximately 25 mm (1 in.) are possible.

The lateral shear transfer behavior for special tank configurations (e.g., shovel bottoms, highly crowned tank bottoms, tanks on grillage) can be unique and are beyond the scope of this Annex.

E.7.7 Local Shear Transfer

Local transfer of the shear from the roof to the shell and the shell of the tank into the base shall be considered. For cylindrical tanks, the peak local tangential shear per unit length shall be calculated by:

$$V_{\max} = \frac{2V}{\pi D} \tag{E.7.7-1}$$

Tangential shear in flat-bottom steel tanks shall be transferred through the welded connection to the steel bottom. The shear stress in the weld shall not exceed 80 % of the weld or base metal yield stress. This transfer mechanism is deemed acceptable for steel tanks designed in accordance with the provisions and S_{DS} < 1.0g.

E.7.8 Connections with Adjacent Structures

Equipment, piping, and walkways or other appurtenances attached to the tank or adjacent structures shall be designed to accommodate the elastic displacements of the tank imposed by design seismic forces amplified by a factor of 3.0 plus the amplified displacement of the other structure.