UNIVERSIDAD NACIONAL DE INGENIERÍA
FACULTAD DE INGENIERÍA CIVIL

HOJA DE CÁLCULO PARA EL DISEÑO DE CIMENTACIONES DE TANQUES DE ALMACENAMIENTO METÁLICOS CIRCULARES

INFORME DE SUFICIENCIA

Para optar el Título Profesional de:

INGENIERO CIVIL

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ÍNDICE

RESUMEN

El presente informe detalla los fundamentos, criterios y procedimientos para el desarrollo de una hoja de cálculo que facilita el diseño de cimentaciones de concreto armado para tanques de acero cilíndricos.

Se desarrolla brevemente en este informe el marco teórico de las cimentaciones, seguido de los criterios tomados para el diseño y desarrollo de la hoja de cálculo, para luego mostrar un ejemplo aplicativo de la hoja de cálculo desarrollada.

Se recoge en este informe los conocimientos de instituciones internacionales y nacionales que permitan un diseño confiable y seguro basado principalmente en los estándares 650 del American Petroleum Institute, edición 11 del año 2007 con adendas del 2008 y 2009, y adaptado con las normas técnicas peruanas NTE vigentes, para todo lo referente a tanques y sus cimentaciones; y complementado con los estándares 318S del American Concrete Institute, edición 2011, para la parte de concreto armado.

LISTA DE CUADROS

LISTA DE FIGURAS

LISTA DE SÍMBOLOS Y DE SIGLAS

A_c : coeficiente de aceleración convectiva

A_i : coeficiente de aceleración impulsiva

b : ancho de la sección de la cimentación

D : diámetro del tanque

E : módulo de elasticidad

E_c : módulo de elasticidad del concreto

E_a : módulo de elasticidad del acero

f'_c : resistencia a la compresión del concreto

F_a : coeficiente de aceleración del lugar

F_v : coeficiente de velocidad del lugar

F_y : esfuerzo de fluencia del acero

G : gravedad específica

g : aceleración de la gravedad

h : altura total de la cimentación

H : altura máxima del contenido del tanque

H_t : altura total del tanque

I : factor de importancia según uso

M_{rw} : momento en cimentación tipo anillo

M_s : momento en cimentación tipo losa

N : resistencia de penetración estándar

Q : factor de escala del sismo máximo considerado

q_d : capacidad portante del suelo

Q_t : carga de nieve sobre el terreno

R : factor de reducción

t_b : espesor de base del tanque

T_c : periodo natural convectivo

T_i : periodo natural impulsivo

t_r : espesor del techo del tanque

t_s : espesor de pared del tanque

V : fuerza cortante en la base

V_{viento} : velocidad el viento

W_c : peso de la masa convectiva

W_f : peso del piso del tanque

W_i : peso de la masa impulsiva

W_p : peso total del contenido del tanque

W_r : peso del techo del tanque

W_s : peso de las paredes del tanque

x_c : altura de centro de gravedad de masa convectiva

x_i : altura de centro de gravedad de masa impulsiva

x_r : altura de centro de gravedad del techo del tanque

x_s : altura de centro de gravedad de las paredes del tanque

Z : factor de zona

INTRODUCCIÓN

CONTEXTO

El presente informe se desarrolla en el marco del curso de actualización de conocimientos para optar el título de ingeniero civil del año 2012. En tal sentido se lleva a cabo este informe aplicando y consolidando los conocimientos y experiencias adquiridos tanto en la universidad como en la vida profesional, en las ramas de construcción, mecánica de suelos y especialmente en estructuras.

El autor se ha desenvuelto en los últimos años en el desarrollo de proyectos industriales principalmente. Es por eso que se elige este tema muy vinculado a obras de minería e hidrocarburos.

ALCANCE

Se ha enfocado un caso puntual del diseño de obras industriales, como son los cimientos de tanques de almacenamiento metálicos circulares. Sin embargo por motivos de tiempo, se limita el presente informe al diseño de cimentaciones de tipo anular de concreto armado.

Para el presente informe no se tomará en cuenta el cálculo de la capacidad portante del suelo, el cual será asumido como dato de entrada, además se limitará para el caso de suelos relativamente buenos ($q_d \geq 2$ kgf/cm^2).

No es alcance de este trabajo los detalles y requerimientos constructivos de este tipo de estructuras, además de problemáticas de casos reales como la limitación de espacio, formas o conexiones que suelen restringir el diseño.

CAPÍTULO I: GENERALIDADES

1.1 ANTECEDENTES

El crecimiento de las industrias mineras y de hidrocarburos en los últimos años demanda una mejor infraestructura para la producción. Es en este contexto que se vive actualmente un gran auge de la ingeniería civil, no sólo por la construcción de edificios de viviendas por el boom inmobiliario que se aprecia en la capital, sino también en las construcciones industriales, especialmente en el sector minero que se desarrolla principalmente en las provincias de la sierra.

En estos proyectos industriales es común encontrar tanques de almacenamiento metálicos a lo largo de sus procesos, para los que debemos diseñar una adecuada cimentación. Por ejemplo, en el caso de plantas de procesos de minerales y algunas de sus facilidades se aprecia que es un elemento que se repite con bastante frecuencia en diversos tamaños, tanto en plantas nuevas como en ampliaciones.

1.2 JUSTIFICACIÓN

Si bien la construcción de estas estructuras no es de las partidas más costosas, se debe tener mucho cuidado ya que cualquier falla es estos procesos, que obligue a detener la producción, sí es bastante costosa.

Con esta hoja de cálculo se pretende facilitar la selección de alternativas para el diseño de estas estructuras, además de ser una herramienta fácil de modificar en caso se requiera. Es decir, en lugar de tener una hoja de cálculo para cada caso específico del problema, se puede tener una hoja más compleja y completa que abarque varios casos.

1.3 PLANTEAMIENTO DEL PROBLEMA

Debido a la gran variedad de casos de suelos, condiciones subterráneas, y condiciones climáticas en los que se puede ubicar la obra, además de algunos requerimientos propios de tanque o su contenido, se diseña cada caso con sus consideraciones y cuidados particulares.

Esta labor suele ser repetitiva en el procedimiento y en muchos casos no se tiene una adecuada metodología, consumiendo muchas horas hombre de alto costo ya que esta labor es desarrollada por profesionales altamente calificados.

1.4 DEFINICIÓN DE OBJETIVOS

Objetivo principal:

El objetivo del presente informe es el desarrollo de una hoja de cálculo que permita:

- Simplificar y reducir el tiempo para el cálculo y diseño de cimentaciones de tanques.
- Facilitar la elaboración de la documentación necesaria para la memoria de cálculo.

Objetivos específicos:

- Desarrollar una hoja de cálculo que facilite el diseño de cimentaciones de concreto armado para tanques de almacenamiento metálicos circulares.
- Aplicar dicha hoja de cálculo a un caso específico ficticio, similar a condiciones reales.

CAPÍTULO II: CIMENTACIONES DE CONCRETO ARMADO

En este capítulo se revisa los fundamentos teóricos necesarios, como conceptos de cimentaciones, y concreto armado para poder diseñar la cimentación.

2.1 CIMENTACIONES

Se define como cimentación al conjunto de estructuras que transmiten las cargas de una edificación o estructura hacia el terreno con la finalidad de sostenerla de manera estable.

2.1.1 Clasificación de cimentaciones

Se puede clasificar las cimentaciones en dos grandes grupos: cimentaciones profundas, y cimentaciones superficiales.

a) Cimentaciones superficiales:

Son aquellas en las cuales la relación Profundidad / ancho (Df/B) es menor o igual a cinco (5), siendo Df la profundidad de la cimentación y B el ancho o diámetro de la misma. Son cimentaciones superficiales las zapatas aisladas, conectadas y combinadas; las cimentaciones continuas (cimientos corridos) y las plateas de cimentación.

b) Cimentaciones profundas:

Son aquellas en las que la relación profundidad /ancho (Df/B) es mayor a cinco (5), siendo Df la profundidad de la cimentación y B el ancho o diámetro de la misma.

Son cimentaciones profundas: los pilotes y micropilotes, los pilotes para densificación, los pilares y los cajones de cimentación.

La cimentación profunda será usada cuando las cimentaciones superficiales generen una capacidad de carga que no permita obtener los factores de seguridad indicados en el Artículo 16 de la NTE E.050 de suelos y cimentaciones o cuando los asentamientos generen asentamientos diferenciales mayores a los indicados en el Artículo 14 también de la NTE E.050. Las cimentaciones profundas se pueden usar también para anclar estructuras contra fuerzas de levantamiento y para colaborar con la resistencia de fuerzas laterales y de volteo. Las cimentaciones profundas pueden además ser requeridas para situaciones especiales tales como suelos expansivos y colapsables o suelos sujetos a erosión.

2.1.2 Tipos de cimentaciones para tanques

a) Cimentación de tierra sin muro anular

Cuando una evaluación técnica de las condiciones del subsuelo, basado en la experiencia y/o un trabajo exploratorio, muestra que la capa de asiento tiene capacidad de soporte adecuada y que los asentamientos serán aceptables, una cimentación adecuada puede ser construida a partir de materiales de la tierra. Los requisitos para el desempeño de las cimentaciones de tierra son los mismos que se exigen a cimientos comunes. En concreto, una fundación tierra debe cumplir con lo siguiente:

a. Proporcionar un plano estable para el soporte del tanque.

b. Limitar el grado de asentamiento global a valores que estén de acuerdo a los permisibles utilizados en el diseño de las conexiones de las tuberías.

c. Proporcionar un drenaje adecuado.

d. No debe asentar excesivamente el perímetro debido al peso de la pared del depósito.

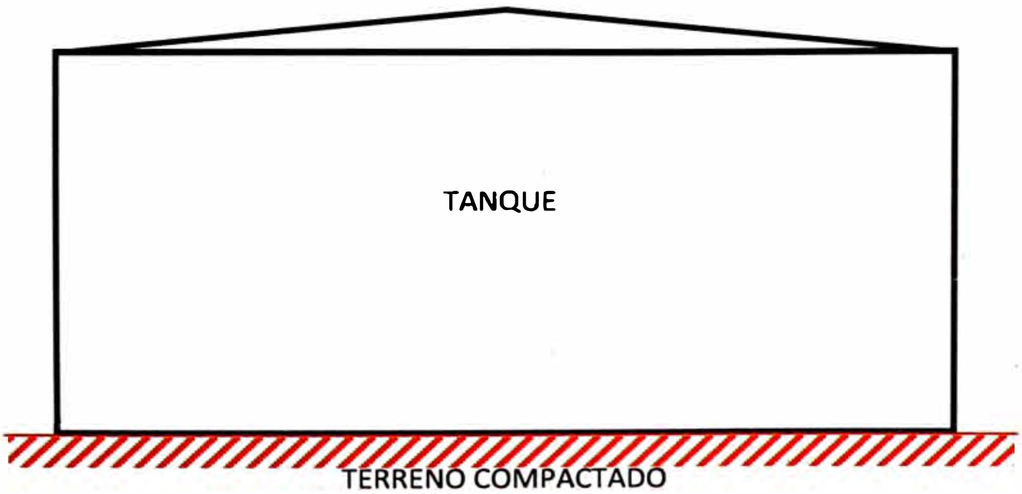


Figura 2.1 Cimentación de tierra sin muro anular

b) Cimentación de tierra con muro anular de concreto

Grandes tanques y tanques con casco pesados o altos y / o con techos auto soportados ejercen una carga sustancial hacia la cimentación debajo del casco. Esto es particularmente importante con respecto a la deformación del casco en los tanques de techo flotante. Se debe utilizar una cimentación con anillo de concreto cuando existe alguna duda sobre la capacidad de la cimentación de soportar la carga del casco directamente. Como una alternativa al anillo de concreto se puede utilizar una un anillo de piedra chancada. Una cimentación con un muro de concreto tiene las siguientes ventajas:

a. Se proporciona una mejor distribución de la carga concentrada del casco para producir una carga de suelo casi uniforme bajo el tanque.

b. Se proporciona un nivel plano y firme de partida para la construcción del casco.

c. Se proporciona una mejor manera de nivelar el tanque, y es capaz de mantener su contorno durante la construcción.

d. Se contiene el relleno bajo la parte inferior del tanque y evita la pérdida de material como resultado de la erosión.

e. Se minimiza la humedad por debajo del tanque.

Una desventaja de los muros anulares de concreto es que no pueden ajustarse con facilidad a los asentamientos diferenciales. Este inconveniente puede dar lugar a altos esfuerzos de flexión en las placas de fondo adyacentes al anillo de concreto.

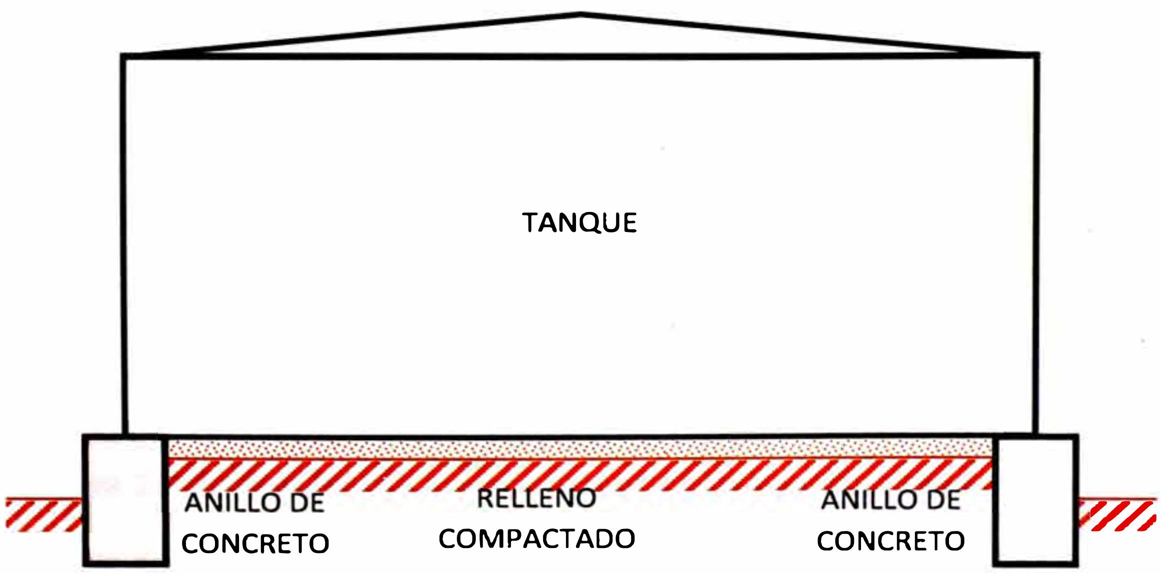


Figura 2.2 Cimentación de tierra con muro anular de concreto

c) Cimentación de tierra con muro anular de piedra chancada o grava

Un anillo de piedra chancada o grava proporcionará un apoyo adecuado a las altas cargas impuestas por el depósito. Una base con anillo de piedra chancada o grava tiene las siguientes ventajas:

a. Se proporciona una mejor distribución de las cargas concentradas del casco para producir una carga de suelo más uniforme bajo el depósito.

b. Se proporciona una mejor manera de nivelar el tanque, y es capaz de preservar su contorno durante la construcción.

c. Se contiene el relleno bajo la parte inferior del tanque y evita la pérdida de material como resultado de la erosión.

d. Se puede acomodar sin problemas a un asentamiento diferencial debido a su flexibilidad.

Una desventaja de los cimientos de tierra con anillo de piedra chancada es que es más difícil de construir al momento de alcanzar las tolerancias y lograr una superficie plana y nivelada para la construcción de la estructura del tanque.

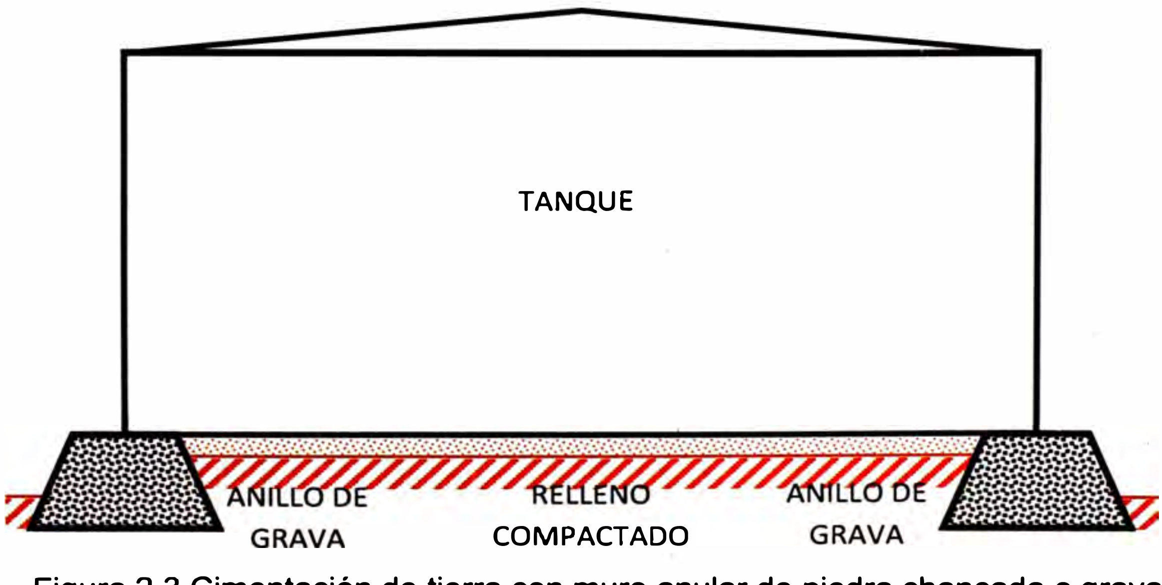


Figura 2.3 Cimentación de tierra con muro anular de piedra chancada o grava

d) Losa de cimentación

Se utilizará una losa de concreto armado cuando las cargas de apoyo transmitidas al suelo deben ser distribuidas sobre un área más grande que el área de depósito o cuando se especifique por el propietario. Puede ser necesario el uso de pilotes debajo de la losa para el soporte apropiado del tanque.

Las cimentaciones para tanques tienen exigencias constructivas y características de diseño que se detallan en los estándares 650 del Instituto Americano del Petróleo (API standard 650).

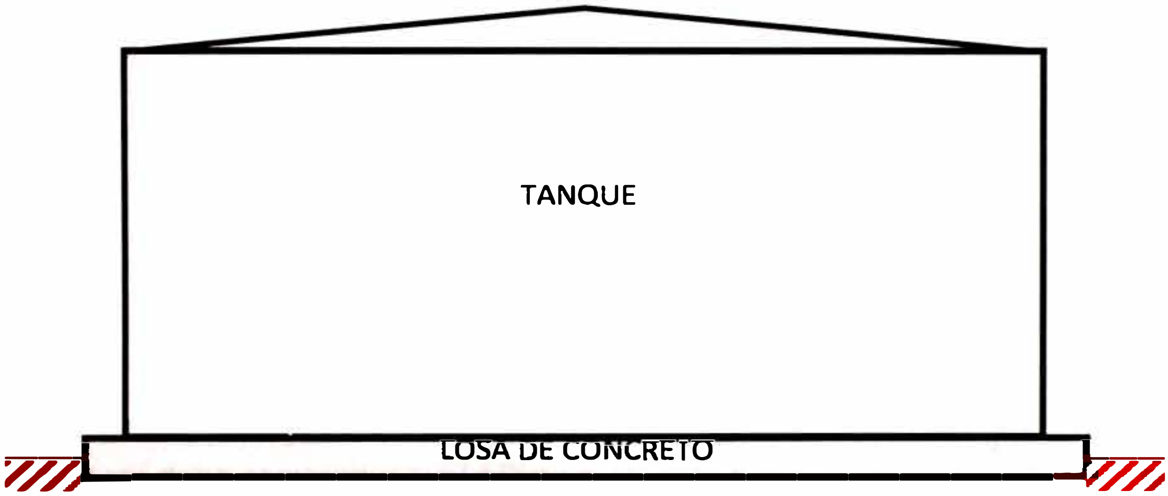


Figura 2.4 Losa de cimentación

2.2 CONCRETO ARMADO

El concreto armado es una mezcla de cemento Portland o cualquier otro cemento hidráulico, agregado fino, agregado grueso y agua, con o sin aditivos, además reforzado con acero. Es utilizado mayormente con fines estructurales.

El concreto armado tiene una serie de exigencias tanto en su fabricación como en su diseño, y normadas por códigos nacionales e internacionales. A continuación se mencionan las principales propiedades del concreto armado.

Resistencia especificada a la compresión del concreto (f'c).Resistencia a la compresión del concreto empleada en el diseño, expresada en MPa. Cuando dicha cantidad esté bajo un signo radical, se quiere indicar sólo la raíz cuadrada del valor numérico, por lo que el resultado está en MPa. La resistencia mínima del concreto estructural, f'c, diseñado y construido de acuerdo con la Norma NTE E.060, no debe ser inferior a 17 MPa.

Módulo elástico del concreto (E_c). Es función principalmente de la resistencia del concreto y de su peso volumétrico.

$$E_c = w^{1.5} 4000\sqrt{f'_c} \qquad \text{(Ec. 2.1)}$$

Donde

w = peso volumétrico del concreto en t/m^3

f'_c = resistencia del concreto en kgf/cm^2

Considerando w = 2.4 t/m^3 se acepta que:

$$E_c = 15000\sqrt{f'_c} \qquad \text{(Ec. 2.2)}$$

Esfuerzo de fluencia del acero F_y. Es el esfuerzo en la gráfica de esfuerzo – deformación donde la deformación continúa aumentando mientras que el esfuerzo permanece casi constante. En nuestro medio el acero más comercial es el grado 60 que tiene un F_y = 4200 kgf/cm^2.

Módulo de elasticidad del acero E_s. es la pendiente de la curva esfuerzo – deformación del acero en la parte elástica (primer tramo recto desde el origen). Se considera que E_s = 2x10^6 kgf/cm^2.

CAPÍTULO III: DISEÑO

3.1 DIAGRAMA DE FLUJO

Esquema general del proceso para el diseño:

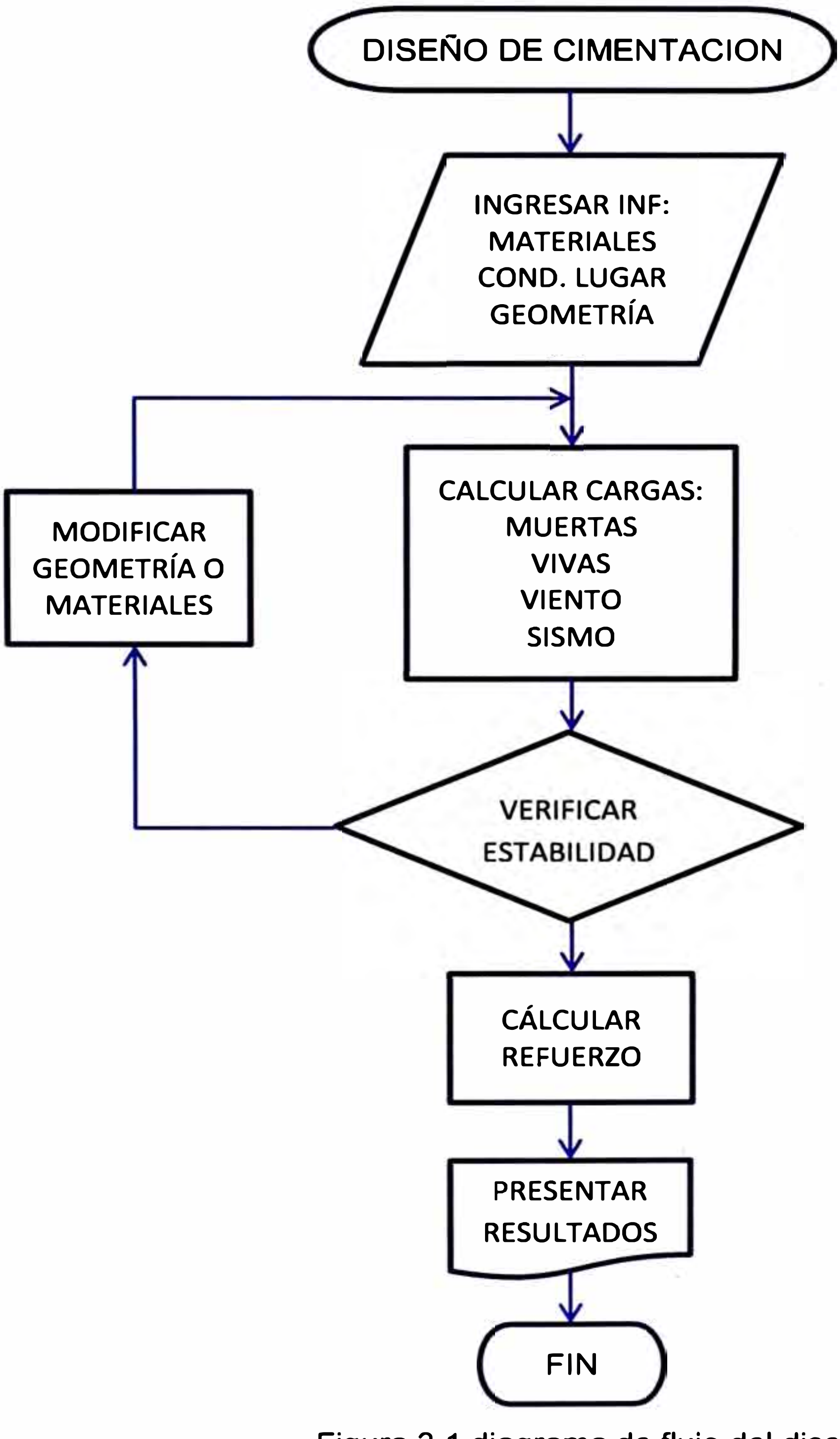


Figura 3.1 diagrama de flujo del diseño

3.2 CARGAS

3.2.1 Pesos unitarios

Para definir las cargas hay que tener en cuenta las propiedades de los materiales. Los pesos específicos de los materiales empleados, en caso de no contar con información exacta del proveedor, se tomarán de la norma de cargas nacional RNE E.020 ANEXO 1 - PESOS UNITARIOS.

Cuadro 3.1 pesos unitarios

Materiales típicos a utilizar:	
Acero	7850 kgf/m^3
Concreto armado	2400 kgf/m^3
Contenidos típicos:	
Agua dulce:	1000 kgf/m^3
Aceites:	930 kgf/m^3
Gasolina:	670 kgf/m^3
Petróleo:	870 kgf/m^3
Ácido nítrico:	1500 kgf/m^3
Ácido sulfúrico:	1800 kgf/m^3
Otros elementos que intervienen:	
Tierra seca	1600 kgf/m^3
Tierra saturada	1800 kgf/m^3
Nieve fresca	100 kgf/m^3
Grava y arenas secas	1600 kgf/m^3

Fuente: NTE E.020

3.2.2 Carga viva de techo

Según la norma de cargas RNE E.020 artículo 7, se debe considerar 100 kgf/m^2 para techos con pendientes de hasta 3°; y para techos con mayor pendiente se reducirá 5 kgf/m^2 por cada grado superior a 3°, con un límite mínimo de 50 kgf/m^2.

0°<pendiente<3° 100 kgf/m2

3°<pendiente<13° [115 – (pendiente x 5)] kgf/m2

Pendiente>13° 50 kgf/m2

3.2.3 Carga de nieve

En caso el tanque se encuentre expuesto a nieve, se debe considerar el efecto de esta carga. Según lo estipulado en la norma RNE E.020 artículo 11, la carga sobre el suelo mínima a considerar es de Q_s = 40 kgf/m^2. Y en el caso de techos será:

a) Para techos a una o dos aguas con inclinaciones menores o iguales a 15° (pendiente ≤ 27%) y para techos curvos con una relación flecha/luz ≤ 0,1 o ángulo vertical menor o igual a 10° (calculado desde el borde hasta el centro) la carga de diseño (Qt), sobre la proyección horizontal, será:

$$Q_t = Q_s \qquad \text{(Ec. 3.1)}$$

b) Para techos a una o dos aguas con inclinaciones comprendidas entre 15° y 30° la carga de diseño (Qt), sobre la proyección horizontal, será:

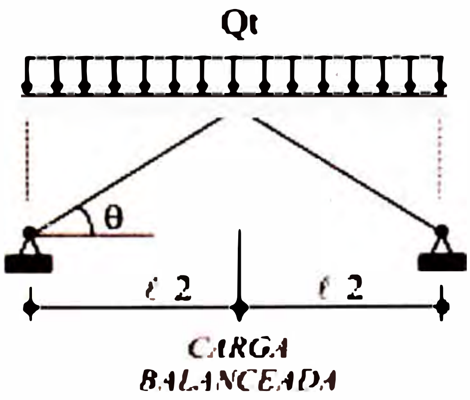
$$Q_t = 0{,}80\ Q_s \qquad \text{(Ec. 3.2)}$$

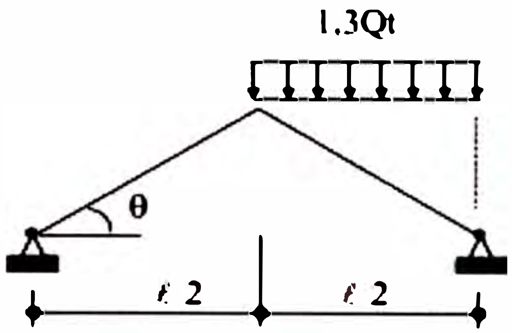
c) Para techos a una o dos aguas con inclinaciones mayores que 30° la carga de diseño (Qt), sobre la proyección horizontal, será:

$$Q_t = C_s\ (0{,}80 Q_s) \qquad \text{(Ec. 3.3)}$$

donde Cs = 1 – 0,025(θ° - 30°), siendo Cs un factor adimensional.

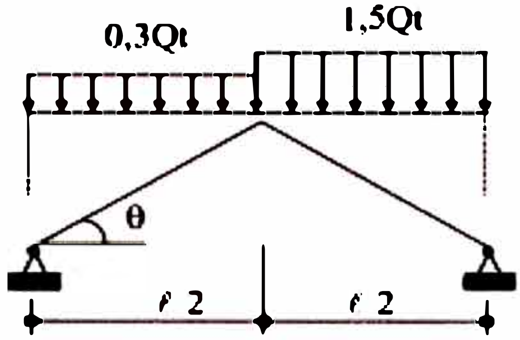
d) Para los techos a dos aguas con inclinaciones mayores que 15° deberán investigarse los esfuerzos internos para las condiciones de carga balanceada y desbalanceada como se indica a continuación:





CARGA DESBALANCEADA

$$\frac{\ell}{2} \leq 6m$$

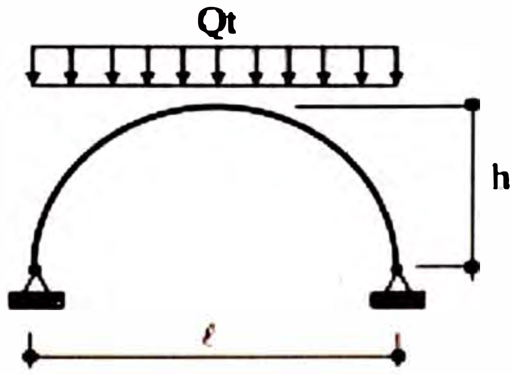


CARGA DESBALANCEADA

$$\frac{\ell}{2} > 6m$$

Figura 3.2 Cargas de nieve

e) Para los techos curvos, dependiendo de la relación h/l, deberán investigarse los esfuerzos internos para las condiciones de cargas balanceada y desbalanceada, que se indica a continuación:



$$\frac{h}{\ell} \leq \frac{1}{10}$$

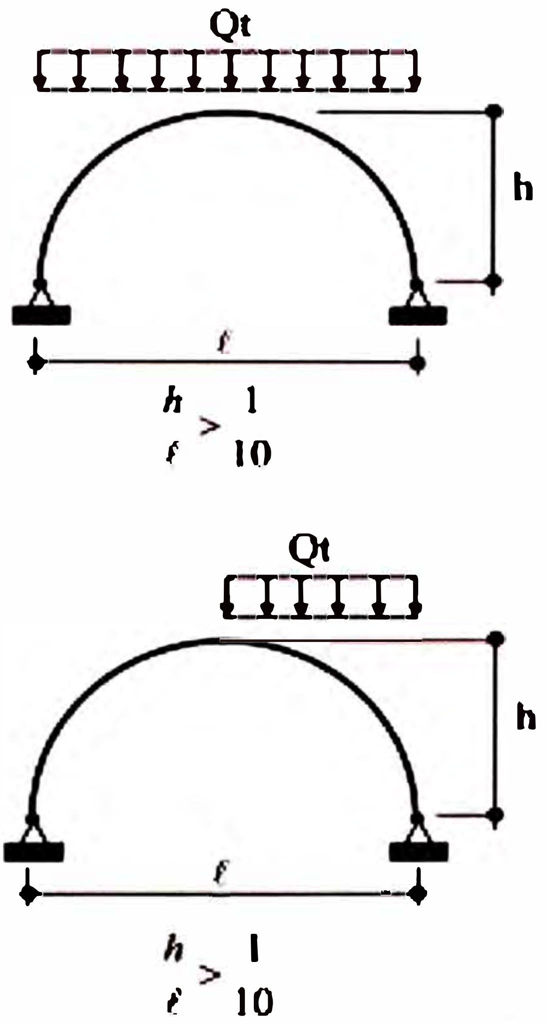


Figura 3.3 Cargas de nieve para techos curvos

3.2.4 Cargas de viento

Clasificación de las edificaciones según efectos del viento

Tipo 1. Edificaciones poco sensibles a las ráfagas y a los efectos dinámicos del viento, tales como edificios de poca altura o esbeltez y edificaciones cerradas con cobertura capaz de soportar las cargas sin variar su geometría.

Tipo 2. Edificaciones cuya esbeltez las hace sensibles a las ráfagas, tales como tanques elevados y anuncios y en general estructuras con una dimensión corta en la dirección del viento. Para este tipo de edificaciones la carga exterior se multiplicará por 1.2.

Tipo 3. Edificaciones que representan problemas aerodinámicos especiales tales como domos, arcos, antenas, chimeneas esbeltas y cubiertas colgantes. Para este tipo de edificaciones las presiones de diseño se determinarán a partir de procedimientos de análisis reconocidos en ingeniería, pero no serán menores que las especificadas para el Tipo 1.

Según la norma de cargas RNE E.020 artículo 12, se puede interpretar un tanque de acero como una estructura sensible a ráfagas. Considerándola como edificación de tipo 2, por lo que la carga que se calcule deberá ser aumentada en 20%.

Velocidad de diseño

La velocidad de diseño del viento hasta 10 m de altura será la velocidad máxima adecuada a la zona de ubicación de la edificación pero no menos de 75 Km/h. La velocidad de diseño del viento en cada altura de la edificación se obtendrá de la siguiente expresión.

$$V_h = V\,(h / 10)^{0.22} \qquad \text{(Ec. 3.4)}$$

Donde:

V_h = velocidad de diseño en la altura h en Km/h

V = velocidad de diseño hasta 10 m de altura en Km/h

h = altura sobre el terreno en metros

Carga exterior de viento

La carga exterior (presión o succión) ejercida por el viento se supondrá estática y perpendicular a la superficie sobre la cual actúa. Se calculará mediante la expresión:

$$P_h = 0{,}005\ C\ V_h^2 \qquad \text{(Ec. 3.5)}$$

Donde:

P_h = presión o succión del viento a una altura h en Kgf/m2

C = factor de forma adimensional indicado en el cuadro 2.2

V_h = velocidad de diseño a la altura h, en Km/h, definida en el artículo anterior

Cuadro 3.2 factores de forma (C) *

CONSTRUCCIÓN	BARLOVENTO	SOTAVENTO
Superficies verticales de edificios	+0,8	-0,6
Anuncios, muros aislados, elementos con una dimensión corta en la dirección del viento	+1,5	
Tanques de agua, chimeneas y otros de sección circular o elíptica	+0,7	
Tanques de agua, chimeneas, y otros de sección cuadrada o rectangular	+2,0	
Arcos y cubiertas cilíndricas con un ángulo de inclinación que no exceda 45°	±0,8	-0,5
Superficies inclinadas a 15° o menos	+0,3-0,7	-0,6
Superficies inclinadas entre 15° y 60°	+0,7-0,3	-0,6
Superficies inclinadas entre 60° y la vertical	+0,8	-0,6
Superficies verticales ó inclinadas (planas ó curvas) paralelas a la dirección del viento	-0,7	-0,7
* El signo positivo indica presión y el negativo succión.		

Fuente NTE E.020

3.2.5 CARGAS DE SÍSMO

El API 650 presenta un procedimiento de diseño sísmico seudo dinámico basado en métodos de análisis de espectro de respuesta y considerando dos modos de respuesta del tanque y su contenido que son el impulsivo y convectivo.

Categoría de la edificación

Las normas API clasifican los tanques de almacenamiento en tres categorías que denominan grupos de uso sísmico (Seismic Use Group, SUG):

SUG III. Grupo de uso sísmico III.

Tanques que dan servicio a instalaciones esenciales para la recuperación luego del terremoto; instalaciones que son esenciales para la vida y salud de la población; o tanques con cantidad considerable de sustancias peligrosas que no cuentan con un adecuado control para prevenir una exposición pública.

SUG II. Grupo de uso sísmico II.

Tanques que almacenan material que supone peligro para la población y carece de controles secundarios de prevención de exposición pública; o aquellos tanques que dan servicio a instalaciones principales.

SUG I. Grupo de uso sísmico I.

Los tanques que no se encuentren en los grupos SUG III Y SUG II.

Tanques de usos múltiples

Se clasificará de acuerdo a los usos y se le asignará el grupo de mayor orden.

Clasificación de suelos

Para el cálculo sísmico por el método del API se empleará la clasificación de suelos del ASCE. El API contempla que se utilice esta clasificación que es aplicable en los Estados Unidos, fuera de este ámbito, utilizando unos factores de seguridad en los parámetros de las aceleraciones espectrales de periodos cortos (S_S) y de de 1 segundo (S_1).

Las clases de sitio se definen como sigue:

Tipo A: Roca dura con velocidad de onda de corte medida de, $\bar{v}_s$ > 1500 m/s (5000 pies/s)

Tipo B: Roca con 760 m/s < $\bar{v}_s$ ≤ 1500 m/s (2500 pies/s < $\bar{v}_s$ ≤ 5000 pies/s)

Tipo C: Suelo muy denso y roca blanda con 360 m/s < $\bar{v}_s$ ≤ 760 m/s (1200 pies/s < $\bar{v}_s$ ≤ 2500 pies/s), o con cualquiera de las siguientes condiciones: N> 50 o $\bar{s}_u$ > 100 kPa (2000 libras por pie cuadrado)

Tipo D: Suelo duro con 180 m/s ≤ $\bar{v}_s$ ≤ 360 m/s (600 pies/s ≤ $\bar{v}_s$ ≤ 1200 pies/s) o con cualquiera de las siguientes condiciones: 15 ≤ N ≤ 50 ó 50 kPa ≤ $\bar{s}_u$ ≤ 100 kPa (1000 libras por pie cuadrado ≤ $\bar{s}_u$ ≤ 2000 libras por pie cuadrado)

Tipo E: Perfil de suelo con $\bar{v}_s$ <180 m/s (600 pies/s) o con cualquiera de las siguientes condiciones: N <15, $\bar{s}_u$ <50 kPa (1000 libras por pie cuadrado), o cualquier perfil con más 3 m (10 pies) de arcilla blanda definida como suelo con PI > 20, w ≥ 40% y $\bar{s}_u$ < 25 kPa (500 libras por pie cuadrado)

Tipo F: Los suelos que requieren evaluaciones de sitio específicas:

1. Los suelos vulnerables a posibles fallos o colapsan bajo cargas sísmicas, tales como suelos licuables, arcillas rápidas y altamente sensibles, suelos plegables débilmente cementados. Sin embargo, puesto que los tanques tienen típicamente un período impulsivo de 0,5 segundos o menos, no se requiere evaluaciones específicas de sitio, pero se recomienda realizarlos para determinar aceleraciones espectrales para suelos licuables. La clase de sitio se puede determinar, en el supuesto de que no se produce licuefacción; y los valores correspondientes de Fa y Fv se determinan a partir de tablas.

2. Turbas y/o arcillas altamente orgánicas (H_S > 3 m [10 pies] de turba y/o arcilla altamente orgánica, donde H = espesor del suelo).

3. Arcillas de muy alta plasticidad (H_S > 8 m [25 pies] con PI > 75).

4. Arcillas muy gruesas, con rigidez suave/media (H_S > 36 m [120 pies])

Los parámetros utilizados para definir la clase de sitio se basan en los 30 m (100 pies) superiores del perfil del sitio. Los perfiles que contienen capas de suelo muy diferentes se subdividirán en las capas designadas por un número que oscila entre 1 y n en la parte inferior donde hay un total de n capas distintas en los primeros 30 m (100 pies). El símbolo i, se refiere a cualquiera de las capas entre 1 y n.

v_{si} = la velocidad de onda de corte en m/s (pies/s),

d_i = el espesor de cualquier capa (entre 0 y 30 m [100 pies]).

$$\bar{v} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \qquad \text{(Ec. 3.6)}$$

Donde:

$$\sum_{i=1}^{n} d_i = 30m$$

N_i = la resistencia de penetración estándar determinado de acuerdo con la norma ASTM D 1586, medida directamente en el campo sin correcciones, y no se tendrá más de 100 golpes/pie.

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}} \qquad \text{(Ec. 3.7)}$$

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_i}} \qquad \text{(Ec. 3.7)}$$

Donde:

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

Utilice sólo d_i y N_i para suelos no cohesivos.

d_s = espesor total de las capas del suelo no cohesivos entre los 30 m (100 pies),

s_{ui} = la resistencia al corte sin drenaje en kPa (libras por pie cuadrado), determinada de acuerdo con ASTM D 2166 o D 2850, y no mayor a 240 kPa (5000 libras por pie cuadrado).

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}} \qquad \text{(Ec. 3.8)}$$

Donde:

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

d_c = espesor total (100 - d_s) de las capas de suelos cohesivos en los 30 m (100 pies),

PI = índice de plasticidad, determinada de acuerdo con ASTM D 4318,

w = contenido de humedad en %, determinada de acuerdo con ASTM D 2216.

Procedimiento para clasificar un sitio:

Paso 1: Verifique que las cuatro categorías de sitio Clase F que requieren evaluación específica del sitio. Si el sitio corresponde a alguna de estas categorías, clasificar el sitio como Sitio de clase F y realizar una evaluación específica del sitio. De lo contrario pasar al siguiente paso.

Paso 2: Comprobar la existencia de arcilla blanda con un espesor total > 3 m (10 pies), donde se define una capa de arcilla blanda por: s_u <25 kPa (500 libras por pie cuadrado); w ≥ 40% y PI > 20. Si se cumplen estos criterios, clasificar el sitio como Sitio Clase E. De no cumplir estas condiciones pasar al siguiente paso.

Paso 3: Clasificar el sitio usando uno de los siguientes tres métodos con $\bar{v}_s$, $\bar{N}$ y $\bar{s}_u$ calculados en todos los casos:

a. $\bar{v}_s$ para los 30 m superiores (100 pies) (método $\bar{v}_s$).

b. $\bar{N}$ para la los 30 m superiores (100 pies) (método $\bar{N}$).

c. $\bar{N}$ para capas de suelo no cohesivos (PI < 20) en los 30 m superiores (100 pies) y el promedio de $\bar{s}_u$ para las capas de suelo cohesivo (PI > 20) en los 30 m superiores (100 ft) (método $\bar{s}_u$).

Cuadro 3.3 Clasificación de suelo

Clasificación de sitio	$\bar{v}_s$ m/s (pies/s)	$\bar{N}$ o $\bar{N}_{ch}$	$\bar{s}_u$ kPa (libra por pie cuadrado)
E	< 180 (< 600)	< 15	< 50 (< 1000)
D	180 – 360 (600 – 1200)	15 to 50	50 – 100 (1000– 2000)
C	360 – 760 (1200 – 2500)	> 50	> 100 (> 2000)
B	760 – 1500 (2500 – 5000)		
A	> 1500 (>5000)		

Nota: Si se utiliza el método $\bar{s}_u$ y los criterios $\bar{N}_{ch}$ y $\bar{s}_u$ difieren, seleccione la categoría con los suelos más desfavorable.

La asignación de la clase del sitio B se basa en la velocidad de la onda de corte para roca. Para roca competente con fractura moderada y a la intemperie, será admitida la estimación de esta velocidad de onda cortante. Para roca altamente fracturada y más erosionada, la velocidad de onda de corte se mide directamente o en el sitio, y se asignará la clase de sitio C.

La asignación de la clase de sitio A se sustentará en cualquiera de las mediciones de velocidad de ondas transversales en el lugar o mediciones de la velocidad de corte de onda en los perfiles del tipo de roca en la misma formación con un grado de desgaste y fractura igual o superior.

Donde se conoce que las condiciones de roca dura son continuos hasta una profundidad de 30 m (100 pies), las medidas de la velocidad de ondas de corte superficiales pueden ser extrapolados para estimar $\bar{v}_s$.

Las clases de sitio A y B no debe ser usado donde hay más de 3 m (10 pies) de suelo entre la superficie de la roca y la parte inferior de la base del tanque.

Periodo de vibración estructural

Periodo convectivo (chapoteo)

El período del primer modo de la ola de chapoteo, en segundos, se calcula por la ecuación siguiente donde K_s es el coeficiente de período de chapoteo:

En unidades del SI:

$$T_c = 1.8K_s\sqrt{D} \quad \text{(Ec. 3.9)}$$

o en unidades americanas:

$$T_c = K_s\sqrt{D} \quad \text{(Ec. 3.10)}$$

$$K_s = \frac{0.578}{\sqrt{tanh\left(\frac{3.68\,H}{D}\right)}} \quad \text{(Ec. 3.11)}$$

Aceleraciones de respuesta de espectro de diseño

Coeficientes de aceleración espectral

Cuando se utilizan métodos de diseño probabilísticos o mapeados, los parámetros de aceleración espectral para el espectro de respuesta de diseño se dan en las ecuaciones siguientes. En zonas fuera de EE.UU., donde los requisitos reglamentarios para determinar el movimiento del terreno para el diseño difieren de los métodos ASCE 7 en los que se basa este método, T_L se tomará como 4 segundos.

En los sitios donde se define sólo la aceleración máxima del terreno, sustituir S_P por S_0 en las ecuaciones E.4.6.1-1 a E.4.6.2-1. El factor de escala, Q, se define como 2/3 para los métodos ASCE 7. Q puede tomarse igual a 1.0 a menos que se defina lo contrario en las exigencias regulatorias en donde no se aplique el ASCE 7. Los coeficientes de amplificación del suelo, F_a y F_v; el valor del factor de importancia I; y los factores de modificación de respuesta ASD, R_{wi} y R_{wc}, serán definidos por los requerimientos regulatorios locales. Si estos valores no están definidos por las normas, se utilizará los valores de este capítulo.

Cuadro 3.4 Coeficiente de aceleración del lugar F_a

Clasificación de sitio	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
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] Se requiere un estudio geotécnico específico del lugar y un análisis de respuesta dinámica.

Cuadro 3.5 Coeficiente de velocidad del lugar F_v

Clasificación de sitio	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
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] Se requiere un estudio geotécnico específico del lugar y un análisis de respuesta dinámica.

Parámetro de aceleración espectral impulsivo A_i:

$$A_i = S_{DS}\left(\frac{I}{R_{wi}}\right) = 2.5\ Q\ F_a\ S_0\left(\frac{I}{R_{wi}}\right) \geq 0.007 \qquad \text{(Ec. 3.12)}$$

Sólo para sitios clase E y F:

$$A_i \geq 0.5\ S_1\left(\frac{I}{R_{wi}}\right) = 0.625\ S_P\left(\frac{I}{R_{wi}}\right) \qquad \text{(Ec. 3.13)}$$

Parámetro de aceleración espectral convectivo A_c:

Cuando $T_C \leq T_L$

$$A_c = KS_{D1}\left(\frac{1}{T_c}\right)\left(\frac{I}{R_{wc}}\right) = 2.5\ K\ Q\ F_a\ S_0\left(\frac{T_s}{T_c}\right)\left(\frac{I}{R_{wi}}\right) \leq A_i \qquad \text{(Ec. 3.14)}$$

Cuando $T_C > T_L$

$$A_c = KS_{D1}\left(\frac{T_L}{T_c^2}\right)\left(\frac{I}{R_{wc}}\right) = 2.5\ K\ Q\ F_a\ S_0\left(\frac{T_s\,T_L}{T_c^2}\right)\left(\frac{I}{R_{wi}}\right) \leq A_i \qquad \text{(Ec. 3.15)}$$

Factores de diseño sísmicos

Fuerzas de diseño

La fuerza lateral de diseño sísmico equivalente será determinada por la relación genérica:

$$F = A\ W_{ef} \qquad \text{(Ec. 3.16)}$$

Donde:

A = coeficiente de aceleración lateral, %g,

W_{ef} = peso efectivo.

Factor de modificación de respuesta

Es un factor de reducción que no deberá ser mayor a los que se indican en el cuadro:

Cuadro 3.6 Factores de reducción

Sistema de anclaje	R_{wi} (impulsivo)	R_{wc} (convectivo)
Auto anclado	3.5	2
Anclaje mecánico	4	2

Auto anclado, cuando el tanque es estable al vuelco o deslizamiento debido a su propio peso. Anclajes mecánicos, cuando es necesario el uso de pernos de anclaje para estabilizar el tanque.

Factor de importancia I

Se define por el grupo de uso sísmico (SUG) según el siguiente cuadro:

Cuadro 3.7 Coeficiente de importancia por uso

SUG	I
I	1.00
II	1.25
III	1.50

3.3 DISEÑO

3.3.1 Cargas de diseño

Los tanques de fondo plano apoyados en tierra que almacenan líquidos deberán estar diseñados para resistir las fuerzas sísmicas calculadas teniendo en cuenta la masa efectiva y presiones de líquido dinámicas en la determinación de las fuerzas laterales equivalentes y la distribución de fuerza lateral. El equivalente de la fuerza de corte lateral en la base se determinará como se indica en los apartados siguientes. La fuerza cortante debida al sismo, resultante de la combinación de los componentes impulsivos y convectivos, se define como la raíz cuadrada de la suma de los cuadrados de estos.

$$V = \sqrt{V_i^2 + V_c^2} \qquad \text{(Ec. 3.17)}$$

Donde:

$$V_i = A_i(W_s + W_r + W_f + W_i) \qquad \text{(Ec. 3.18)}$$

$$V_c = A_i\, W_c \qquad \text{(Ec. 3.19)}$$

Producto del peso efectivo

Peso impulsivo efectivo

Cuando D/H ≥ 1.333

$$W_i = \frac{\tanh\left(0.866\,\frac{D}{H}\right)}{0.866\,\frac{D}{H}}\, W_p \qquad \text{(Ec. 3.20)}$$

Cuando D/H < 1.333

$$W_i = \left(1.0 - 0.218\ \frac{D}{H}\right)\ W_p \qquad \text{(Ec. 3.21)}$$

Peso convectivo efectivo

$$W_c = 0.230\ \frac{D}{H} \tanh\left(\frac{3.67\ H}{D}\right)\ W_p \qquad \text{(Ec. 3.22)}$$

Centro de acción de las fuerzas laterales efectivas

Para el caso de cimentaciones de anillos de concreto

Altura de acción del peso impulsivo

Cuando D/H ≥ 1.333

$$X_i = 0.375\ H \qquad \text{(Ec. 3.23)}$$

Cuando D/H < 1.333

$$X_i = \left(0.5 - 0.094\ \frac{D}{H}\right)\ H \qquad \text{(Ec. 3.24)}$$

Altura de acción del peso convectivo

$$X_c = \left(1.0 - \frac{\cosh\left(\frac{3.67\ H}{D}\right) - 1}{\frac{3.67\ H}{D}\ senh\left(\frac{3.67\ H}{D}\right)}\right)\ H \qquad \text{(Ec. 3.25)}$$

Para el caso de cimentaciones tipo losa

Altura de acción del peso impulsivo

Cuando D/H ≥ 1.333

$$X_{is} = 0.375\left(1.0 + 1.333\left(\frac{0.866\ \frac{D}{H}}{\tanh\left(0.866\ \frac{D}{H}\right)} - 1.0\right)\right)\ H \qquad \text{(Ec. 3.26)}$$

Cuando D/H < 1.333

$$X_{is} = \left(0.5 - 0.060\ \frac{D}{H}\right)\ H \qquad \text{(Ec. 3.27)}$$

Altura de acción del peso convectivo

$$X_{cs} = \left(1.0 - \frac{\cosh\left(\frac{3.67\,H}{D}\right) - 1.937}{\frac{3.67\,H}{D}\,\operatorname{senh}\left(\frac{3.67\,H}{D}\right)}\right) H \qquad \text{(Ec. 3.28)}$$

Momento de volteo

Para cimentaciones de anillo de concreto

$$M_{rw} = \sqrt{[A_i(W_iX_i + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_c)]^2} \qquad \text{(Ec. 3.29)}$$

Para cimentaciones tipo losa

$$M_s = \sqrt{[A_i(W_iX_{is} + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_{cs})]^2} \qquad \text{(Ec. 3.30)}$$

3.3.2 Estabilidad

Estabilidad de volteo del tanque

Para tanques con anclaje mecánico la relación de estabilidad de volteo es la siguiente:

$$\frac{0.5\,D\,[W_p + W_f + W_T + W_{fd} + W_g]}{M_{s\ o\ rw}} \geq 2 \qquad \text{(Ec. 3.31)}$$

Estabilidad al volteo de la cimentación

Los momentos que actúan debido a las fuerzas de empuje lateral del relleno (teoría de Rankine), deben ser menores que los momentos resistentes debidos al peso propio del cimiento y de la sobrecarga del tanque.

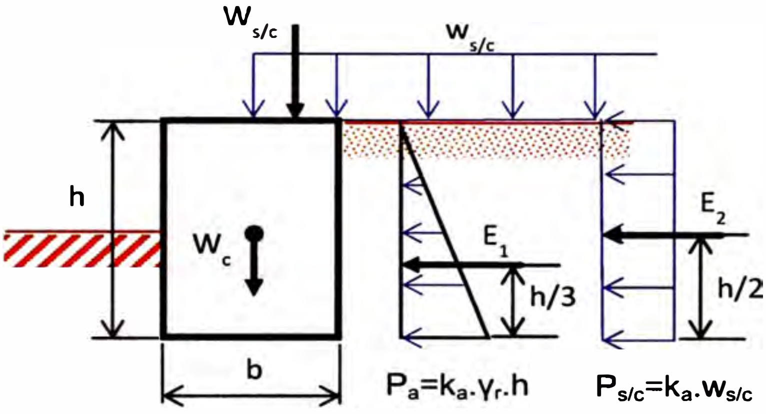


Figura 3.4 empuje de tierra

Empuje activo

$$E_1 = \frac{k_a \, \gamma_r \, h^2}{2} \qquad \text{(Ec. 3.32)}$$

Empuje debido a sobre carga

$$E_2 = k_a \, w_{s/c} \qquad \text{(Ec. 3.33)}$$

Donde

$$k_a = 1 - sen(\emptyset) \qquad \text{(Ec. 3.34)}$$

Estabilidad por capacidad de carga

La presión máxima no puede exceder a la capacidad portante del terreno determinado mediante un estudio de suelos.

CAPÍTULO IV: EJEMPLO DE APLICACIÓN

4.1 DATOS DEL EJEMPLO

Para el ejemplo se tomará un tanque de acero circular con techo cónico sobre una cimentación de concreto armado tipo anular; para el almacenamiento de agua; ubicado en la sierra del departamento de La Libertad.

Las dimensiones del tanque son:

Diámetro de 3.80 m

Altura total de 4.50 m

Altura efectiva del agua 4.10 m

Ángulo del techo de 5°

Espesores del techo y del fondo del tanque de 8.38 mm

Espesores del pared del tanque de 6.35 mm

Se va a asumir las condiciones de nieve y viento mínimas para el reglamento.

Se asumirá también una capacidad portante del terreno de 2 kgf/cm^2.

Debido a la importancia medioambiental y costo del proceso se va a asignar el mayor factor de importancia de edificación.

Como prediseño se ha asignado las dimensiones de la cimentación de 0.75 m x 0.75 m de sección, y el mismo diámetro del tanque como eje.

4.2 HOJA DE CÁLCULO

La hoja de cálculo ha sido desarrollada en Excel versión 2010. Los cálculos auxiliares y comentarios están del lado derecho de la hoja, y las instrucciones en la parte superior, de cada hoja, siempre fuera del área de impresión.

Se debe mantener tener activada la opción de cálculo iterativo para que la hoja funcione correctamente.

Además se han añadido hojas de apoyo dentro del mismo libro para facilitar las decisiones del diseñador que se muestran en los anexos.

Las celdas sombreadas son las que se deben añadir manualmente.

Se deben ingresar los datos de los materiales y parámetros del estudio de suelo, además de las dimensiones de la cimentación tentativas a verificar.

ESPECIFICACIONES DE LOS MATERIALES

CIMENTACIÓN

Relleno

Peso Unitario	γ_s	=	1800.00 kgf/m³
Ángulo de Fricción Interna	Ø	=	30.00 °
Fricción Entre la Base y el Tanque	μ	=	0.40

Concreto

Esfuerzo de Compresión	f'c	=	210.00 kgf/cm²
Peso Unitario	γ_c	=	2400.00 kgf/m³

Acero de Refuerzo

Esfuerzo de Fluencia	Fy	=	4200.00 kgf/cm²

TANQUE

Acero

Peso específico	γ_s	=	7850.00 kgf/m³
Esfuerzo de Fluencia	Fy	=	36.00 ksi

Líquido

Peso específico	γ_w	=	1000.00 kgf/m³
Gravedad específica	G	=	1.00

Importancia del Uso del Tanque: grupo de uso sísmico SUG según API 650

SUG III	I	=	1.50

CONDICIONES DE LUGAR

Zonificación Sísmica Según Norma Técnica E.030 Diseño Sismoresistente

Zona 3	Z	=	0.40

Tipo de suelo (clasificación ASCE en API 650)

Tipo D

Nieve

Carga de nieve según NTE E.020 (0 si es que no aplica)	Q_s	=	40.00 kgf/m²

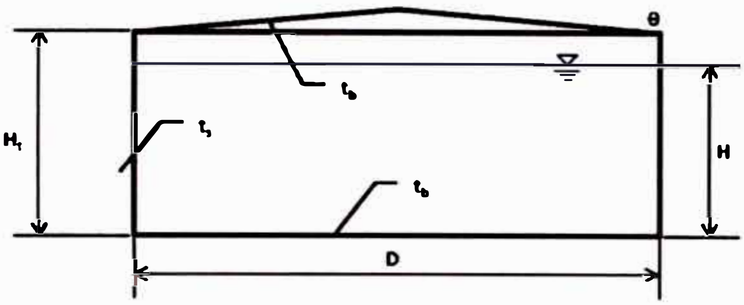
Viento

Velocidad máxima de viento del lugar (no menor a 75 km/h)	V_{viento}	=	75.00 km/h

Suelo de Fundación

Capacidad Portante	q_a	=	2.00 kgf/cm²

CARACTERÍSTICAS GEOMÉTRICAS DEL TANQUE



Geometría

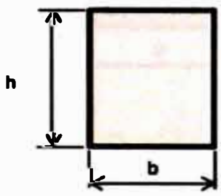
Diámetro nominal	D	=	3.80 m
Altura del tanque	H_1	=	4.50 m
Altura del líquido	H	=	4.10 m
Ángulo del techo	θ	=	5.00 °

Espesores del tanque

Espesor del techo del tanque	t_r	=	8.38 mm
Espesor de la pared del tanque	t_s	=	6.38 mm
Espesor del fondo del tanque	t_b	=	8.38 mm

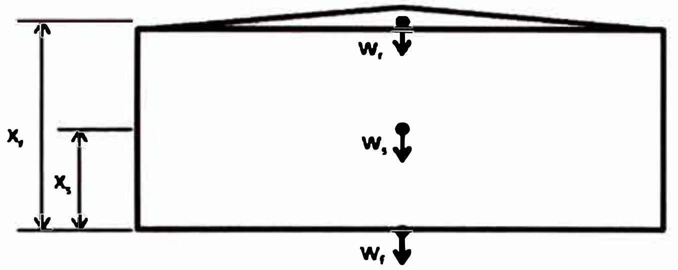
CARACTERÍSTICAS GEOMÉTRICAS DE LA CIMENTACIÓN

Sección de la cimentación



h	=	0.75 m
b	=	0.75 m

CARGAS DEL TANQUE

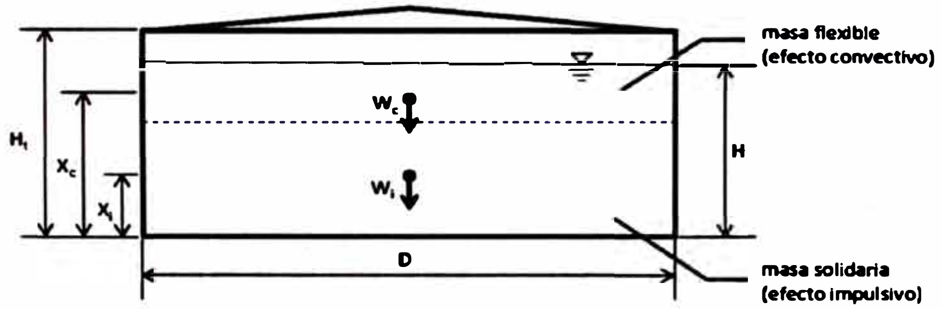


Pesos

Peso del techo del tanque	W_r	=	831.71 kgf
Peso de las paredes del tanque	W_s	=	2816.19 kgf
Peso del piso del tanque	W_f	=	746.05 kgf

Distancia de centro de acción de pesos

Altura del centro de gravedad del techo	X_r	=	4.58 m
Altura del centro de gravedad de la paredes (H_t / 2)	X_s	=	2.25 m



Pesos del contenido del tanque

Peso Total del Contenido	W_p	=	46498.71 kgf
Peso Efectivo Impulsivo (API 650)	W_i	=	37103.70 kgf
Peso Efectivo Convectivo (API 650)	W_c	=	9904.96 kgf

Distancia de Aplicación de Pesos efectivos

Distancia de aplicación del peso Impulsivo (API 650)	X_i	=	1.69 m
Distancia de aplicación del peso Convectivo (API 650)	X_c	=	3.10 m

CÁLCULO DE LA FUERZA SÍSMICA

Momento de Volteo para cimentaciones de anillo de concreto (API 650)

$$M_{rw} = \sqrt{[A_i(W_iX_i + W_sX_s + W_rX_r)]^2 + [A_c(W_cX_c)]^2}$$

Parámetro de aceleración máxima de diseño	S_p	=	0.40 g
Parámetro de respuesta de aceleración espectral (periodos cortos)	S_s	=	1.00 g
Parámetro de respuesta de aceleración espectral (periodos 1s)	S_1	=	0.50 g
Parámetro de respuesta de aceleración espectral (periodos 0s)	S_0	=	0.50 g
Coeficiente de aceleración del lugar (Tabla E-1 API 650)	F_a	=	1.10
Coeficiente de velocidad del lugar (Tabla E-2 API 650)	F_v	=	1.50
Periodo convectivo	T_c	=	2.040 s
Periodo natural del primer modo	T_s	=	0.682 s
Aceleración Espectral Impulsiva	A_i	=	0.516 g
Aceleración Espectral Convectiva	A_c	=	0.516 g
Momento de Volteo	M_{rw}	=	40821.19 kgf.m

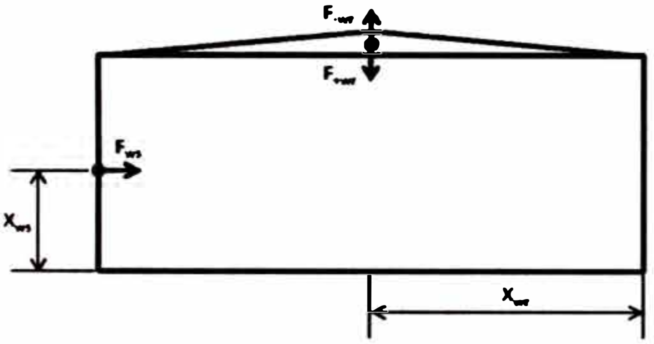
Fuerza Cortante (API 650)

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

Fuerza Cortante Impulsiva	V_i	=	21397.23 kgf
Fuerza Cortante Convectiva	V_c	=	5107.24 kgf
Fuerza Cortante en la Base	V	=	21998.31 kgf

CÁLCULO DE LA FUERZA DE VIENTO

Presión de viento lateral según NTE E.020	P_s	=	23.63 kgf/m²
Presión de viento en techo según NTE E.020	P_{+r}	=	10.13 kgf/m²
Succión de viento en techo según NTE E.020	P_{-r}	=	-23.63 kgf/m²
Área lateral proyectada expuesta al viento	A_s	=	17.10 m²
Área de techo expuesta al viento	A_r	=	11.38 m²



	X_{ws}	=	2.25 m
	X_{wr}	=	1.90 m
Fuerza de viento lateral	F_{ws}	=	403.99 kgf
Fuerza de presión de viento en techo	F_{+wr}	=	115.27 kgf
Fuerza de succión de viento en techo	F_{-wr}	=	-268.96 kgf
Momento de volteo por viento	M_w	=	1419.99 kgf.m

ESTABILIDAD DEL TANQUE

Verificación de volteo por sismo

$$\frac{0.5\,D\,[W_p + W_f + W_T + W_{fd} + W_g]}{M_{s\,o\,rw}} \geq 2$$

Peso de la estructura $W_s + W_r$	W_T	=	3647.91 kgf
Peso de la cimentación	W_{fd}	=	16116.37 kgf
Peso del relleno sobre la cimentación	W_g	=	0 kgf

3.12 > 2 ES ESTABLE A MOMENTO DE VOLTEO DE SISMO

Verificación de volteo por viento

$$\frac{0.5\,D\,[W_T]}{M_w} \geq 1.5$$

4.88 > 1.5 ES ESTABLE A MOMENTO DE VOLTEO DE VIENTO

DISEÑO DE LA CIMENTACIÓN

Datos de los Materiales

Concreto armado

Resistencia del concreto	f'_c	=	210.00 kgf/cm²
Peso específico del concreto	γ_c	=	2.40 t/m³
Módulo de elasticidad concreto	E_c	=	217371 kgf/cm³

Acero de refuerzo

Esfuerzo de fluencia del acero	F_y	=	4200.00 kgf/cm²
Módulo de elasticidad del acero	E_s	=	2000000 Kg/cm³

Contenido del Tanque

Peso específico del líquido	γ_w	=	1.00 t/m³

Material de relleno

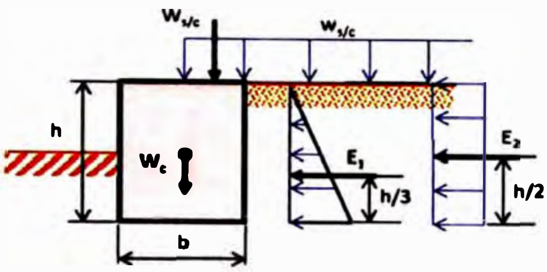
Peso específico del material relleno	γ_s	=	1.80 t/m³
Ángulo de fricción	$\varnothing$	=	30 °

Capacidad portante del terreno

Capacidad portante del Terreno	q_d	=	20.00 t/m²

ESTABILIDAD DE LA CIMENTACIÓN

Verificación de volteo



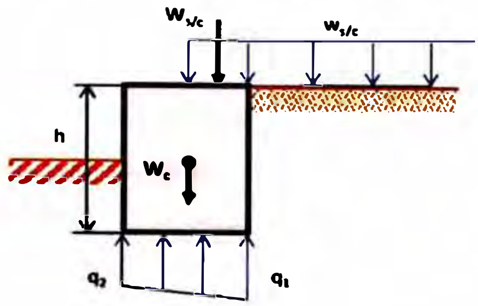
b	=	0.75 m
h	=	0.75 m
L	=	1.00 m

se analizará sólo para 1m

Sobrecarga	$w_{s/c}$	=	4.42 t/m2
Coeficiente de presion activa de suelos	K_a	=	0.50
Empuje activo	E_1	=	0.25 t
Empuje de sobrecarga	E_2	=	1.66 t
Momento debido a empuje activo	M_1	=	0.06 t.m
Momento debido a empuje de sobrecarga	M_2	=	0.62 t.m
Peso propio de la cimentación	W_c	=	1.35 t
Sobrecarga sobre el cimiento	$W_{s/c}$	=	1.66 t
Momento debido a peso propio	M_{wc}	=	0.51 t.m
Momento debido a sobrecarga	$M_{s/c}$	=	0.93 t.m
Momento total actuante	M_a	=	0.69 t.m
Momento total resistente	M_r	=	1.44 t.m
Factor de Seguridad contra Volteo	FSV	=	2.10

FSV = 2.10 > 2 ES ESTABLE A MOMENTO DE VOLTEO

Verificación de capacidad de carga del terreno local



q_1 = 8.00 t/m

q_2 = 0.02 t/m

Excentricidad de la carga   e = 0.124 m

q_1 = 8.00 < 20.00 q_d

q_2 = 0.02 < 20.00 q_d

ES ESTABLE POR CAPACIDAD DEL TERRENO

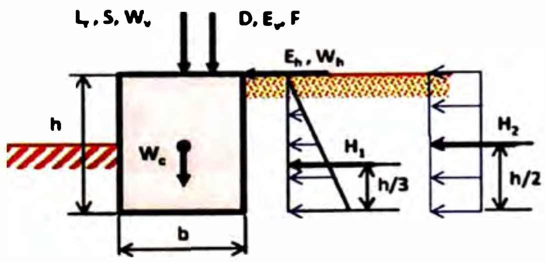
CONCRETO ARMADO

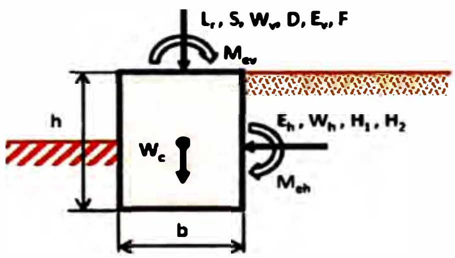
El análisis se realizará para una sección crítica de la cimentación de 1m de largo.

COMBINACIONES DE CARGA según ACI 318S y NTE E.060

Combinación	D	L	Lr	S	R	W	E	F	H
1	1.4							1.4	
2	1.2	1.6	0.5					1.2	1.6
3	1.2	1.6		0.5				1.2	1.6
4	1.2	1.6			0.5			1.2	1.6
5	1.2	1	1.6					1.2	
6	1.2	1		1.6				1.2	
7	1.2	1			1.6			1.2	
8	1.2		1.6			0.8		1.2	
9	1.2		1.6			-0.8		1.2	
10	1.2			1.6		0.8		1.2	
11	1.2			1.6		-0.8		1.2	
12	1.2				1.6	0.8		1.2	
13	1.2				1.6	-0.8		1.2	
14	1.2	1	0.5			1		1.2	
15	1.2	1	0.5			-1		1.2	
16	1.2	1	0.5	0.5		1		1.2	
17	1.2	1	0.5	0.5		-1		1.2	
18	1.2	1	0.5		0.5	1		1.2	
19	1.2	1	0.5		0.5	-1		1.2	
20	1.2	1		0.2			1	1.2	
21	1.2	1		0.2			-1	1.2	
22	0.9					1			1.6
23	0.9					-1			0.9
24	0.9						1	0.9	1.6
25	0.9						-1	0.9	0.9

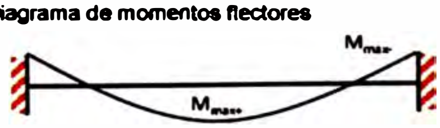
<u>Carga muerta</u>			
Peso del techo del tanque	W_r	=	786.35 kgf
Peso de las paredes del tanque	W_s	=	2816.19 kgf
Peso del piso del tanque	W_f	=	746.05 kgf
Peso total del tanque	W_{tanque}	=	4348.60 kgf
Peso del tanque distribuido sobre la cimentación	w_{tanque}	=	364.26 kgf/m
Peso de la cimentación	W_c	=	1.35 t
	D	=	0.36 t
<u>Carga viva</u>	L	=	0.00 t
<u>Carga viva de techo</u>	Lr	=	0.09 t
<u>Carga de nieve</u>	S	=	0.04 t
<u>Carga de viento</u>	W_v	=	0.37 t
	W_h	=	0.20 t
<u>Carga de sismo</u>	E_v	=	0.71 t
	E_h	=	1.84 t
<u>Carga del fluido</u>	F	=	1.54 t
<u>Carga de empuje de tierra</u>	H_1	=	0.25 t
	H_2	=	1.66 t





Suponiendo porción de cimiento no apoyado (asentamiento o falla del suelo) y suponiendo además que es recto.

Diagrama de momentos flectores



l	=	2.98	m
$M_{v,max+}$	=	3.43	t.m
$M_{v,max-}$	=	1.71	t.m
$M_{h,max+}$	=	3.64	t.m
$M_{h,max-}$	=	1.82	t.m

Combinación	vert	vert-W_c	horiz	horiz-H_2	M_{ev}	M_{eh}	M
1	4.55	2.66	0.00	0.00	0.50	0.00	0.50
2	3.94	2.32	3.06	0.41	0.43	0.05	0.48
3	3.92	2.30	3.06	0.41	0.43	0.05	0.48
4	3.90	2.28	3.06	0.41	0.43	0.05	0.48
5	4.04	2.42	0.00	0.00	0.43	0.00	0.43
6	3.96	2.34	0.00	0.00	0.43	0.00	0.43
7	3.90	2.28	0.00	0.00	0.43	0.00	0.43
8	4.34	2.72	0.16	0.16	0.43	-0.06	0.37
9	3.74	2.12	-0.16	-0.16	0.43	0.06	0.49
10	4.26	2.64	0.16	0.16	0.43	-0.06	0.37
11	3.66	2.04	-0.16	-0.16	0.43	0.06	0.49
12	4.20	2.58	0.16	0.16	0.43	-0.06	0.37
13	3.60	1.98	-0.16	-0.16	0.43	0.06	0.49
14	4.32	2.70	0.20	0.20	0.43	-0.08	0.35
15	3.57	1.95	-0.20	-0.20	0.43	0.08	0.50
16	4.34	2.72	0.20	0.20	0.43	-0.08	0.35
17	3.59	1.97	-0.20	-0.20	0.43	0.08	0.50
18	4.32	2.70	0.20	0.20	0.43	-0.08	0.35
19	3.57	1.95	-0.20	-0.20	0.43	0.08	0.50
20	4.62	3.00	1.84	1.84	0.56	-0.69	-0.13
21	3.20	1.58	-1.84	-1.84	0.29	0.69	0.99
22	1.92	0.70	3.26	0.61	0.06	-0.03	0.04
23	1.17	-0.05	1.52	0.03	0.06	0.10	0.17
24	3.64	2.42	4.90	2.25	0.45	-0.64	-0.19
25	2.22	1.00	-0.12	-1.61	0.19	0.72	0.91
	vert	vert-W_c	horiz	horiz-H_2	M_{ev}	M_{eh}	M
máximo	4.62	3.00	4.90	2.25	0.56	0.72	0.99
mínimo	1.17	-0.05	-1.84	-1.84	0.06	-0.69	-0.19

Aplastamiento del concreto

$U / A_1 \leq \phi(0.85\ f'_c)$

En la parte baja del cimiento — U = 6.16 t/m^2

En el apoyo del tanque — U = 8.00 t/m^2

U= 8.00 < 1160.3 — RESISTE AL APLASTAMIENTO

Diseño por flexión

Factor de reducción por flexión — ϕ = 0.90

Cuantía mínima: — ρ_{min} = 0.0018

β_1 = 0.85

Cuantía balanceada — ρ_b = 0.0213

Cuantía máxima — ρ_{max} = 0.0106

Cálculo de acero

Acero transversal

Distancia de la fibra en compresion al refuerzo — d = 69.21 cm

$$A_s = \frac{M_u}{\emptyset f_y (d - \frac{a}{2})}$$ — A_s = 0.38 cm^2

$$a = \frac{A_s F_y}{0.85 f'_c b_{un}}$$ — a = 0.09 cm

ρ = 0.0001

$\rho_{min} < \rho < \rho_{max}$

Utilizar acero mínimo — A_s = 12.46 cm^2

Se colocará como estribos — Ø 5/8 @ 14.00 cm

A_s = 14.14 cm^2

Acero longitudinal inferior

Distancia de la fibra en compresion al refuerzo — d = 69.21 cm

$$A_s = \frac{M_u}{\emptyset f_y (d - \frac{a}{2})}$$ — A_s = 1.32 cm^2

$$a = \frac{A_s F_y}{0.85 f'_c b_{un}}$$ — a = 0.31 cm

ρ = 0.0002

$\rho_{min} < \rho < \rho_{max}$

Utilizar acero mínimo — A_s = 9.34 cm^2

Se colocará — 5 Ø 5/8

A_s = 9.90 cm^2

Acero longitudinal superior

Distancia de la fibra en compresion al refuerzo — d = 69.21 cm

$$A_s = \frac{M_u}{\emptyset f_y (d - \frac{a}{2})}$$ — A_s = 0.66 cm^2

$$a = \frac{A_s F_y}{0.85 f'_c b_{un}}$$ — a = 0.15 cm

ρ = 0.0001

$\rho_{min} < \rho < \rho_{max}$

Utilizar acero mínimo — A_s = 9.34 cm^2

Se colocará — 5 Ø 5/8

A_s = 9.90 cm^2

acero lateral

Distancia de la fibra en compresion al refuerzo d = 69.21 cm

$$A_s = \frac{M_u}{\emptyset f_y (d - \frac{a}{2})}$$ A_s = 1.40 cm^2

$$a = \frac{A_s F_y}{0.85 f'_c b_{un}}$$ a = 0.33 cm

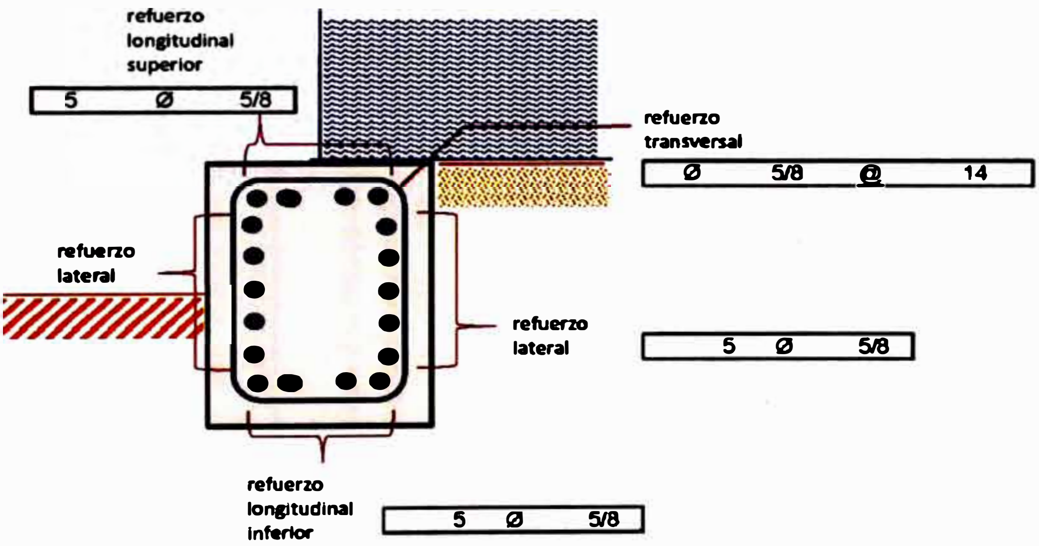
ρ = 0.0002

ρ_{min} < ρ < ρ_{max}

Utilizar acero mínimo A_s = 9.34 cm^2

Se colocará 5 Ø 5/8

A_s = 9.90 cm^2



Se obtiene finalmente la distribución del acero para la cimentación.

4.3 CUADROS DE APOYO DE LA HOJA DE CÁLCULO

◉ SUG III **Grupo de uso sísmico III.**

Tanques que dan servicio a instalaciones esenciales para la recuperación luego del terremoto; instalaciones que son esenciales para la vida y salud de la población; o tanques con cantidad considerable de sustancias peligrosas que no cuentan con un adecuado control para prevenir una exposición pública.

○ SUG II **Grupo de uso sísmico II.**

Tanques que almacenan material que supone peligro para la población y carece de controles secundarios de prevención de exposición pública; o aquellos tanques que dan servicio a instalaciones principales.

○ SUG I **Grupo de uso sísmico I.**

Los tanques que no se encuentren en los grupos SUG III Y SUG II.

Los tanques de usos múltiples se clasificará de acuerdo a los usos y se le asignará el grupo de mayor orden.

Cuadro 4.1 Clasificación por uso sísmico

Cuadro 4.2 Zonificación sísmica

ZONIFICACIÓN SÍSMICA SEGÚN NORMA TÉCNICA E.030 DISEÑO SISMORESISTENTE

zona	Z
1	0.15
2	0.30
3	0.40

ZONA	3
Z	0.40

Zona 1

1. Departamento de Loreto. Provincias de Mariscal Ramón Castilla, Maynas y Requena.
2. Departamento de Ucayali. Provincia de Purús.
3. Departamento de Madre de Dios. Provincia de Tahuamanú.

Zona 2

1. Departamento de Loreto. Provincias de Loreto, Alto Amazonas, Ucayali y Datem del Marañón
2. Departamento de Amazonas. Todas las provincias.
3. Departamento de San Martín. Todas las provincias.
4. Departamento de Huánuco. Todas las provincias.
5. Departamento de Ucayali. Provincias de Coronel Portillo, Atalaya y Padre Abad.
6. Departamento de Pasco. Todas las provincias.
7. Departamento de Junín. Todas las provincias.
8. Departamento de Huancavelica. Provincias de Acobamba, Angaraes, Churcampa, Tayacaja y Huancavelica.
9. Departamento de Ayacucho. Provincias de Sucre, Huamanga, Huanta, Vilcashuaman y La Mar.
10. Departamento de Apurimac. Todas las provincias.
11. Departamento de Cusco. Todas las provincias.
12. Departamento de Madre de Dios. Provincias de Tambopata y Manú.
13. Departamento de Puno. Todas las provincias.

Zona 3

1. Departamento de Tumbes. Todas las provincias.
2. Departamento de Piura. Todas las provincias.
3. Departamento de Cajamarca. Todas las provincias.
4. Departamento de Lambayeque. Todas las provincias.
5. Departamento de La Libertad. Todas las provincias.
6. Departamento de Ancash. Todas las provincias.
7. Departamento de Lima. Todas las provincias.
8. Provincia Constitucional del Callao.
9. Departamento de Ica. Todas las provincias.
10. Departamento de Huancavelica. Provincias de Castrovirreyna y Huaytará.
11. Departamento de Ayacucho. Provincias de Cangallo, Huanca Sancos, Lucanas, Víctor Fajardo, Parinacochas y Paucar del Sara Sara.
12. Departamento de Arequipa. Todas las provincias.
13. Departamento de Moquegua. Todas las provincias.
14. Departamento de Tacna. Todas las provincias.

Cuadro 4.3 Clasificación de suelo

Clasificación de sitio	OBSERVACIONES	$\bar{v}_s$ m/s (pies/s)	$\bar{N}$ o $\bar{N}_{ch}$	$\bar{s}_u$ kPa (libra por pie cuadrado)
Tipo F	Cuatro casos: 1. Los suelos vulnerables a posibles fallos o colapsan bajo cargas sísmicas, tales como suelos licuables, arcillas rápidas y altamente sensibles, suelos plegables débilmente cementados. Sin embargo, puesto que los tanques tienen típicamente un período impulsivo de 0,5 segundos o menos, no se requiere evaluaciones específicas de sitio, pero se recomienda realizarlos para determinar aceleraciones espectrales para suelos licuables. 2. Turbas y/o arcillas altamente orgánicas (H_S > 3 m [10 pies] de turba y/o arcilla altamente orgánica, donde H = espesor del suelo). 3. Arcillas de muy alta plasticidad (H_S > 8 m [25 pies] con PI > 75). 4. Arcillas muy gruesas, con rigidez suave/media (H_S > 36 m [120 pies])	Ver nota	Ver nota	Ver nota
Tipo E	Suelo con las propiedades de tabla o perfil con más 3 m de arcilla blanda	< 180 (< 600)	< 15	< 50 (< 1000)
Tipo D	Suelo duro	180 – 360 (600 – 1200)	15 to 50	50 – 100 (1000– 2000)
Tipo C	Suelo muy denso y roca blanda	360 – 760 (1200 – 2500)	> 50	> 100 (> 2000)
Tipo B	Roca	760 – 1500 (2500 – 5000)		
Tipo A	Roca dura	> 1500 (>5000)		

Nota: Ver procedimiento para clasificar el tipo de suelo (pág. 27, Procedimiento para clasificar un sitio, Cap. III). La hoja de cálculo del presente informe no trabaja para el tipo F.

CAPITULO V: CONCLUSIONES Y RECOMENDACIONES

5.1 CONCLUSIONES

1. Es posible automatizar gran parte del trabajo de diseño incluyendo las decisiones de alternativas que dependen de la comparación de parámetros.
2. Simplificar el diseño requiere de una ardua labor, sin embargo si el trabajo es repetitivo, a largo plazo resulta un gran ahorro de tiempo y esfuerzo.
3. El cálculo para cualquier cimentación de tipo anular dentro de los límites de los parámetros del presente informe se puede realizar y revisar en pocos minutos.
4. Se puede apreciar también en el ejemplo que la combinación de cargas que exige más a la estructura es por esfuerzos sísmicos.

5.2 RECOMENDACIONES

1. Es preferible invertir tiempo en la elaboración, actualización y desarrollo de hojas de cálculo que faciliten el trabajo.
2. Si se desea optimizar el diseño tal vez sea recomendable reducir el efecto convectivo reduciendo el posible oleaje de la masa del líquido. Por ejemplo reduciendo la altura o aumentando la relación D/H.
3. Esta hoja de cálculo aún tiene aspectos que mejorar o implementarse, y con el tiempo necesitará de actualizaciones para adaptarse a los cambios en las normas. Sin embargo los criterios y conceptos básicos no cambian. Sugiero al lector que encuentre este informe de utilidad, no conformarse con copiar esta hoja de cálculo, sino crear una mejor rescatando lo mejor de este trabajo. Por ejemplo, espero pronto poder ampliar la hoja para poder diseñar tanto cimentaciones de concreto tipo anular como del tipo losa.
4. El presente informe se desarrolla en un ámbito netamente académico y para condiciones específicas. Es responsabilidad del profesional que decida utilizar esta información, la actualización, revisión y uso adecuado del presente documento y archivos digitales adjuntos.

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ANEXOS

- ANEXO 1: API STD 650 APPENDIX B – RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION OF FOUNDATIONS FOR ABOVEGROUND OIL STORAGE TANKS (RECOMENDACIONES PARA EL DISEÑO Y CONSTRUCCIÓN DE CIMENTACIONES PARA TANQUES DE ALMACENAMIENTO DE ACEITE APOYADOS SOBRE EL SUELO)

- ANEXO 2: API STD 650 APPENDIX E – SEISMIC DESIGN OF STORAGE TANKS (DISEÑO SÍSMICO DE TANQUES DE ALMACENAMIENTO)

ANEXO 1: API STD 650 APPENDIX B – RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION OF FOUNDATIONS FOR ABOVEGROUND OIL STORAGE TANKS (RECOMENDACIONES PARA EL DISEÑO Y CONSTRUCCIÓN DE CIMENTACIONES PARA TANQUES DE ALMACENAMIENTO DE ACEITE APOYADOS SOBRE EL SUELO)

APPENDIX B—RECOMMENDATIONS FOR DESIGN AND CONSTRUCTION OF FOUNDATIONS FOR ABOVEGROUND OIL STORAGE TANKS

B.1 Scope

B.1.1 This appendix provides important considerations for the design and construction of foundations for aboveground steel oil storage tanks with flat bottoms. Recommendations are offered to outline good practice and to point out some precautions that should be considered in the design and construction of storage tank foundations.

B.1.2 Since there is a wide variety of surface, subsurface, and climatic conditions, it is not practical to establish design data to cover all situations. The allowable soil loading and the exact type of subsurface construction to be used must be decided for each individual case after careful consideration. The same rules and precautions shall be used in selecting foundation sites as would be applicable in designing and constructing foundations for other structures of comparable magnitude.

B.2 Subsurface Investigation and Construction

B.2.1 At any tank site, the subsurface conditions must be known to estimate the soil bearing capacity and settlement that will be experienced. This information is generally obtained from soil borings, load tests, sampling, laboratory testing, and analysis by an experienced geotechnical engineer familiar with the history of similar structures in the vicinity. The subgrade must be capable of supporting the load of the tank and its contents. The total settlement must not strain connecting piping or produce gauging inaccuracies, and the settlement should not continue to a point at which the tank bottom is below the surrounding ground surface. The estimated settlement shall be within the acceptable tolerances for the tank shell and bottom.

B.2.2 When actual experience with similar tanks and foundations at a particular site is not available, the following ranges for factors of safety should be considered for use in the foundation design criteria for determining the allowable soil bearing pressures. (The owner or geotechnical engineer responsible for the project may use factors of safety outside these ranges.)

a. From 2.0 to 3.0 against ultimate bearing failure for normal operating conditions.

b. From 1.5 to 2.25 against ultimate bearing failure during hydrostatic testing.

c. From 1.5 to 2.25 against ultimate bearing failure for operating conditions plus the maximum effect of wind or seismic loads.

B.2.3 Some of the many conditions that require special engineering consideration are as follows:

a. Sites on hillsides, where part of a tank may be on undisturbed ground or rock and part may be on fill or another construction or where the depth of required fill is variable.

b. Sites on swampy or filled ground, where layers of muck or compressible vegetation are at or below the surface or where unstable or corrosive materials may have been deposited as fill.

c. Sites underlain by soils, such as layers of plastic clay or organic clays, that may support heavy loads temporarily but settle excessively over long periods of time.

d. Sites adjacent to water courses or deep excavations, where the lateral stability of the ground is questionable.

e. Sites immediately adjacent to heavy structures that distribute some of their load to the subsoil under the tank sites, thereby reducing the subsoil's capacity to carry additional loads without excessive settlement.

f. Sites where tanks may be exposed to flood waters, possibly resulting in uplift, displacement, or scour.

g. Sites in regions of high seismicity that may be susceptible to liquefaction.

h. Sites with thin layers of soft clay soils that are directly beneath the tank bottom and that can cause lateral ground stability problems.

B.2.4 If the subgrade is inadequate to carry the load of the filled tank without excessive settlement, shallow or superficial construction under the tank bottom will not improve the support conditions. One or more of the following general methods should be considered to improve the support conditions:

a. Removing the objectionable material and replacing it with suitable, compacted material.

b. Compacting the soft material with short piles.

c. Compacting the soft material by preloading the area with an overburden of soil. Strip or sand drains may be used in conjunction with this method.

d. Stabilizing the soft material by chemical methods or injection of cement grout

e. Transferring the load to a more stable material underneath the subgrade by driving piles or constructing foundation piers. This involves constructing a reinforced concrete slab on the piles to distribute the load of the tank bottom.

f. Constructing a slab foundation that will distribute the load over a sufficiently large area of the soft material so that the load intensity will be within allowable limits and excessive settlement will not occur.

g. Improving soil properties by vibro-compaction, vibro-replacement, or deep dynamic-compaction.

h. Slow and controlled filling of the tank during hydrostatic testing. When this method is used, the integrity of the tank may be compromised by excessive settlements of the shell or bottom. For this reason, the settlements of the tank shall be closely monitored. In the event of settlements beyond established ranges, the test may have to be stopped and the tank releveled.

B.2.5 The fill material used to replace muck or other objectionable material or to build up the grade to a suitable height shall be adequate for the support of the tank and product after the material has been compacted. The fill material shall be free of vegetation, organic matter, cinders, and any material that will cause corrosion of the tank bottom. The grade and type of fill material shall be capable of being compacted with standard industry compaction techniques to a density sufficient to provide appropriate bearing capacity and acceptable settlements. The placement of the fill material shall be in accordance with the project specifications prepared by a qualified geotechnical engineer.

B.3 Tank Grades

B.3.1 The grade or surface on which a tank bottom will rest should be constructed at least 0.3 m (1 ft) above the surrounding ground surface. This will provide suitable drainage, help keep the tank bottom dry, and compensate for some small settlement that is likely to occur. If a large settlement is expected, the tank bottom elevation shall be raised so that the final elevation above grade will be a minimum of 150 mm (6 in.) after settlement.

B.3.2 There are several different materials that can be used for the grade or surface on which the tank bottom will rest. To minimize future corrosion problems and maximize the effect of corrosion prevention systems such as cathodic protection, the material in contact with the tank bottom should be fine and uniform. Gravel or large particles shall be avoided. Clean washed sand 75 mm 100 mm (3 in. 4 in.) deep is recommended as a final layer because it can be readily shaped to the bottom contour of the tank to provide maximum contact area and will protect the tank bottom from coming into contact with large particles and debris. Large foreign objects or point contact by gravel or rocks could cause corrosion cells that will cause pitting and premature tank bottom failure.

During construction, the movement of equipment and materials across the grade will mar the graded surface. These irregularities should be corrected before bottom plates are placed for welding.

Adequate provisions, such as making size gradients in sublayers progressively smaller from bottom to top, should be made to prevent the fine material from leaching down into the larger material, thus negating the effect of using the fine material as a final layer. This is particularly important for the top of a crushed rock ringwall.

Note: For more information on tank bottom corrosion and corrosion prevention that relates to the foundation of a tank, see API RP 651.

B.3.3 Unless otherwise specified by the Purchaser, the finished tank grade shall be crowned from its outer periphery to its center at a slope of 1 in. in 10 ft. The crown will partly compensate for slight settlement, which is likely to be greater at the center. It will also facilitate cleaning and the removal of water and sludge through openings in the shell or from sumps situated near the shell. Because crowning will affect the lengths of roof-supporting columns, it is essential that the tank Manufacturer be fully informed of this feature sufficiently in advance. (For an alternative to this paragraph, see B.3.4.)

B.3.4 As an alternative to B.3.3, the tank bottom may be sloped toward a sump. The tank Manufacturer must be advised as required in B.3.3.

B.4 Typical Foundation Types

B.4.1 EARTH FOUNDATIONS WITHOUT A RINGWALL

B.4.1.1 When an engineering evaluation of subsurface conditions that is based on experience and/or exploratory work has shown that the subgrade has adequate bearing capacity and that settlements will be acceptable, satisfactory foundations may be constructed from earth materials. The performance requirements for earth foundations are identical to those for more extensive foundations. Specifically, an earth foundation should accomplish the following:

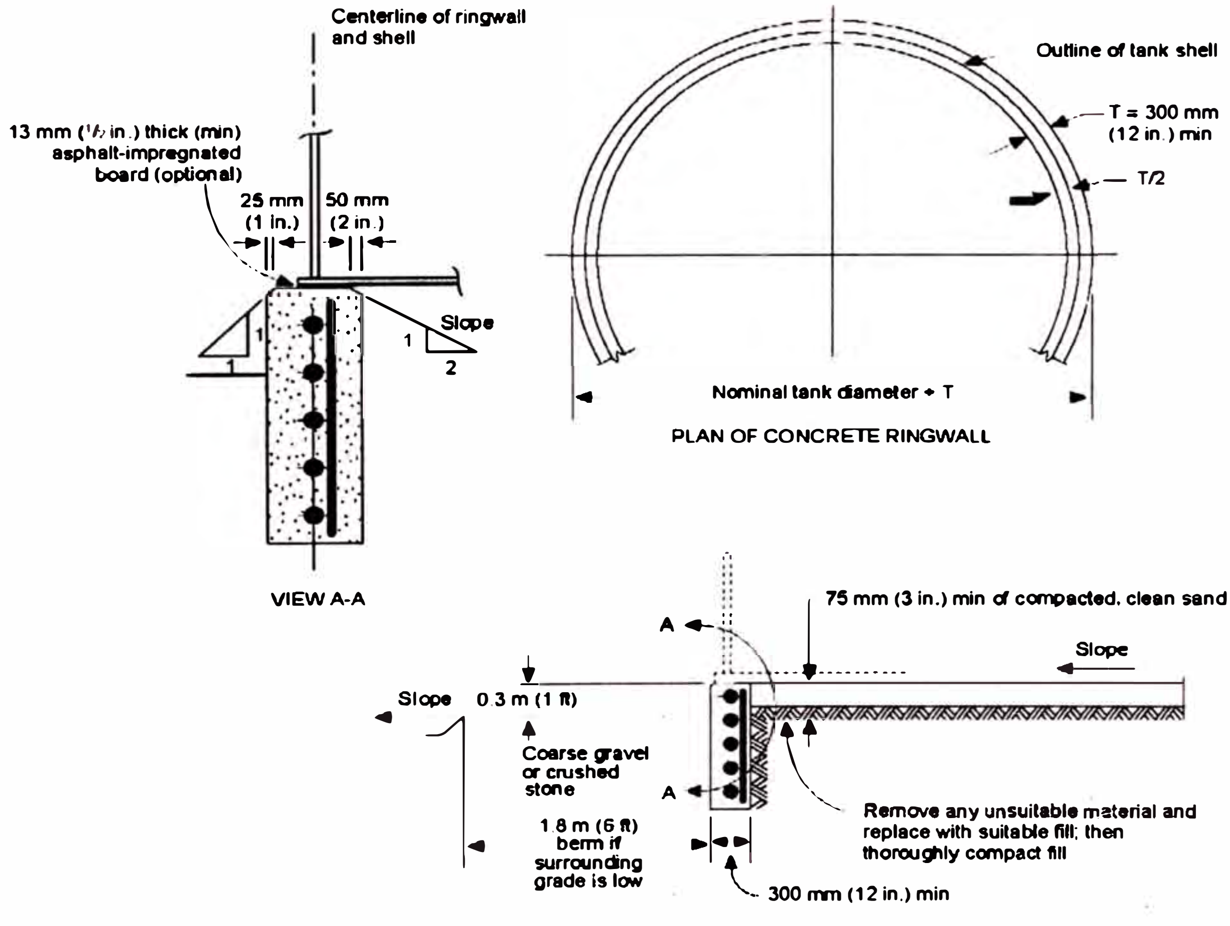
a. Provide a stable plane for the support of the tank.

b. Limit overall settlement of the tank grade to values compatible with the allowances used in the design of the connecting piping.

c. Provide adequate drainage.

d. Not settle excessively at the perimeter due to the weight of the shell wall.

B.4.1.2 Many satisfactory designs are possible when sound engineering judgment is used in their development. Three designs are referred to in this appendix on the basis of their satisfactory long-term performance. For smaller tanks, foundations can consist of compacted crushed stone, screenings, fine gravel, clean sand, or similar material placed directly on virgin soil. Any unstable material must be removed, and any replacement material must be thoroughly compacted. Two recommended designs that include ringwalls are illustrated in Figures B-1 and B-2 and described in B.4.2 and B.4.3.



Notes:

1. See B.4.2.3 for requirements for reinforcement.
2. The top of the concrete ringwall shall be smooth and level. The concrete strength shall be at least 20 MPa (3000 lbf/in.^{2}) after 28 days. Reinforcement splices must be staggered and shall be lapped to develop full strength in the bond. If staggering of laps is not possible, see ACI 318 for additional development requirements.
3. Ringwalls that exceed 300 mm (12 in.) in width shall have rebars distributed on both faces.
4. See B.4.2.2 for the position of the tank shell on the ringwall.

Figure B-1—Example of Foundation with Concrete Ringwall

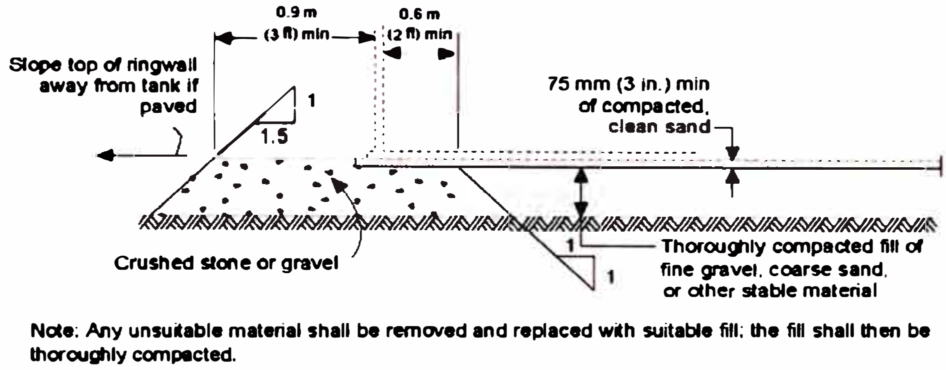


Figure B-2—Example of Foundation with Crushed Stone Ringwall

B.4.2 EARTH FOUNDATIONS WITH A CONCRETE RINGWALL

B.4.2.1 Large tanks and tanks with heavy or tall shells and/or self-supported roofs impose a substantial load on the foundation under the shell. This is particularly important with regard to shell distortion in floating-roof tanks. When there is some doubt whether a foundation will be able to carry the shell load directly, a concrete ringwall foundation should be used. As an alternative to the concrete ringwall noted in this section, a crushed stone ringwall (see B.4.3) may be used. A foundation with a concrete ringwall has the following advantages:

a. It provides better distribution of the concentrated load of the shell to produce a more nearly uniform soil loading under the tank.
b. It provides a level, solid starting plane for construction of the shell.
c. It provides a better means of leveling the tank grade, and it is capable of preserving its contour during construction.
d. It retains the fill under the tank bottom and prevents loss of material as a result of erosion.
e. It minimizes moisture under the tank.

A disadvantage of concrete ringwalls is that they may not smoothly conform to differential settlements. This disadvantage may lead to high bending stresses in the bottom plates adjacent to the ringwall.

B.4.2.2 When a concrete ringwall is designed, it shall be proportioned so that the allowable soil bearing is not exceeded. The ringwall shall not be less than 300 mm (12 in.) thick. The centerline diameter of the ringwall should equal the nominal diameter of the tank; however, the ringwall centerline may vary if required to facilitate the placement of anchor bolts or to satisfy soil bearing limits for seismic loads or excessive uplift forces. The depth of the wall will depend on local conditions, but the depth must be sufficient to place the bottom of the ringwall below the anticipated frost penetration and within the specified bearing strata. As a minimum, the bottom of the ringwall, if founded on soil, shall be located 0.6 m (2 ft) below the lowest adjacent finish grade. Tank foundations must be constructed within the tolerances specified in 7.5.5. Recesses shall be provided in the wall for flush-type cleanouts, drawoff sumps, and any other appurtenances that require recesses.

B.4.2.3 A ringwall should be reinforced against temperature changes and shrinkage and reinforced to resist the lateral pressure of the confined fill with its surcharge from product loads. ACI 318 is recommended for design stress values, material specifications, and rebar development and cover. The following items concerning a ringwall shall be considered:

a. The ringwall shall be reinforced to resist the direct hoop tension resulting from the lateral earth pressure on the ringwall's inside face. Unless substantiated by proper geotechnical analysis, the lateral earth pressure shall be assumed to be at least 50% of the vertical pressure due to fluid and soil weight. If a granular backfill is used, a lateral earth pressure coefficient of 30% may be used.

b. The ringwall shall be reinforced to resist the bending moment resulting from the uniform moment load. The uniform moment load shall account for the eccentricities of the applied shell and pressure loads relative to the centroid of the resulting soil pressure. The pressure load is due to the fluid pressure on the horizontal projection of the ringwall inside the shell.

c. The ringwall shall be reinforced to resist the bending and torsion moments resulting from lateral, wind, or seismic loads applied eccentrically to it. A rational analysis, which includes the effect of the foundation stiffness, shall be used to determine these moments and soil pressure distributions.

d. The total hoop steel area required to resist the loads noted above shall not be less than the area required for temperature changes and shrinkage. The hoop steel area required for temperature changes and shrinkage is 0.0025 times the vertical cross-sectional area of the ringwall or the minimum reinforcement for walls called for in ACI 318, Chapter 14.

e. For ringwalls, the vertical steel area required for temperature changes and shrinkage is 0.0015 times the horizontal cross-sectional area of the ringwall or the minimum reinforcement for walls called for in ACI 318, Chapter 14. Additional vertical steel may be required for uplift or torsional resistance. If the ring foundation is wider than its depth, the design shall consider its behavior as an annular slab with flexure in the radial direction. Temperature and shrinkage reinforcement shall meet the ACI 318 provisions for slabs. (See ACI 318, Chapter 7.)

f. When the ringwall width exceeds 460 mm (18 in.), using a footing beneath the wall should be considered. Footings may also be useful for resistance to uplift forces.

g. Structural backfill within and adjacent to concrete ringwalls and around items such as vaults, undertank piping, and sumps requires close field control to maintain settlement tolerances. Backfill should be granular material compacted to the density and compacting as specified in the foundation construction specifications. For other backfill materials, sufficient tests shall be conducted to verify that the material has adequate strength and will undergo minimal settlement.

h. If the tank is designed and constructed for elevated temperature service, see B.6.

B.4.3 EARTH FOUNDATIONS WITH A CRUSHED STONE AND GRAVEL RINGWALL

B.4.3.1 A crushed stone or gravel ringwall will provide adequate support for high loads imposed by a shell. A foundation with a crushed stone or gravel ringwall has the following advantages:

a. It provides better distribution of the concentrated load of the shell to produce a more nearly uniform soil loading under the tank.

b. It provides a means of leveling the tank grade, and it is capable of preserving its contour during construction.

c. It retains the fill under the tank bottom and prevents loss of material as a result of erosion.

d. It can more smoothly accommodate differential settlement because of its flexibility.

A disadvantage of the crushed stone or gravel ringwall is that it is more difficult to construct it to close tolerances and achieve a flat, level plane for construction of the tank shell.

B.4.3.2 For crushed stone or gravel ringwalls, careful selection of design details is necessary to ensure satisfactory performance. The type of foundation suggested is shown in Figure B-2. Significant details include the following:

a. The 0.9 m (3 ft) shoulder and berm shall be protected from erosion by being constructed of crushed stone or covered with a permanent paving material.

b. Care shall be taken during construction to prepare and maintain a smooth, level surface for the tank bottom plates.

c. The tank grade shall be constructed to provide adequate drainage away from the tank foundation.

d. The tank foundation must be true to the specified plane within the tolerances specified in 7.5.5.

B.4.4 SLAB FOUNDATIONS

B.4.4.1 When the soil bearing loads must be distributed over an area larger than the tank area or when it is specified by the owner, a reinforced concrete slab shall be used. Piles beneath the slab may be required for proper tank support.

B.4.4.2 The structural design of the slab, whether on grade or on piles, shall properly account for all loads imposed upon the slab by the tank. The reinforcement requirements and the design details of construction shall be in accordance with ACI 318.

B.5 Tank Foundations for Leak Detection

Appendix I provides recommendations on the construction of tank and foundation systems for the detection of leaks through the bottoms of storage tanks.

B.6 Tank Foundations for Elevated Temperature Service

he design and construction of foundations for tanks operating at elevated temperatures [> 93°C (200°F)] should address the following considerations.

a. When subjected to elevated operating temperatures, an unanchored tank may tend to move in one or more directions over time. This movement must be accommodated in the design of the tank fittings and attachments.

b. Elevated temperature service may evaporate moisture in the soil supporting the tank and lead to increased, and possibly non-uniform, settlement. Such settlement may include differential settlement between the ringwall and soil under the tank bottom immediately adjacent to the ringwall resulting from non-uniform shrinkage of the soil with respect to the stone or concrete ringwall.

c. In cases where there is high groundwater table, elevated temperatures may vaporize groundwater and generate undesirable steam.

d. Attachments between the tank and the foundation must accommodate the thermal expansion and contraction of the tank without resulting in unacceptable stress levels.

e. The elevated temperature must be accounted for in the design of concrete ringwall foundations. The ringwall is subject to a moment due to the higher temperature at the top of the ringwall with respect to the temperature at the bottom of the ringwall. If not adequately accounted for in the design of the ringwall, this moment can lead to cracking of the concrete foundation and loss of tank support.

ANEXO 2: API STD 650 APPENDIX E – SEISMIC DESIGN OF STORAGE TANKS (DISEÑO SÍSMICO DE TANQUES DE ALMACENAMIENTO)

APPENDIX E—SEISMIC DESIGN OF STORAGE TANKS

Part I—Provisions

E.1 Scope

This appendix 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 appendix 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 appendix 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 appendix 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 appendix are derived from ASCE 7, which is based on a maximum considered earthquake ground motion defined as the motion due to an event with a 2% probability of exceedance within a 50-year period (a recurrence interval of approximately 2,500 years). 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 appendix 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 appendix. 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 appendix 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 appendix. Seismic design of floating roofs is beyond the scope of this appendix.

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 appendix.

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

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 (0.4 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 maximum considered earthquake (MCE): The most severe earthquake ground motion considered in this appendix.

E.2.1.4 mechanically-anchored tank: Tanks that have anchor bolts, straps or other mechanical devices to anchor the tank to the foundation.

E.2.1.5 self-anchored tank: Tanks that use the inherent stability of the self-weight of the tank and the stored product to resist overturning forces.

E.2.1.6 site class: A classification assigned to a site based on the types of soils present and their engineering properties as defined in this appendix.

E.2.2 NOTATIONS

A	Lateral acceleration coefficient, %g
A_c	Convective design response spectrum acceleration coefficient, %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 coefficient, %g
C_d	Deflection amplification factor, $C_d = 2$
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_{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	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% 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 D 1586
$\overline{N}$	Average field standard penetration test for the top 30 m (100 ft)

n_A	Number of equally-spaced anchors around the tank circumference
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.)
P_A	Anchorage attachment design load, N (lbf)
P_{AB}	Anchor design load, N (lbf)
Pf	Overturning bearing force based on the maximum longitudinal shell compression at the base of shell, N/m (lbf/ft)
PI	Plasticity index, ASTM D 4318
Q	Scaling factor from the MCE 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	Mapped, maximum considered earthquake, 5% damped, spectral response acceleration parameter at a period of zero seconds (peak ground acceleration for a rigid structure), %g
S_1	Mapped, maximum considered earthquake, 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, %g
S_{DS}	The design, 5% damped, spectral response acceleration parameter at short periods (T = 0.2 seconds) based on ASCE 7 methods, %g
S_P	Design level peak ground acceleration parameter for sites not addressed by ASCE methods
S_S	Mapped, maximum considered earthquake, 5% damped, spectral response acceleration parameter at short periods (0.2 sec), %g
s_u	Undrained shear strength, ASTM D 2166 or ASTM D 2850
$\bar{s}_u$	Average undrained shear strength in top 30 m (100 ft)
t	Thickness of the shell ring under consideration, 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 less corrosion allowance, 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	$0.2\,F_vS_1/F_aS_S$
T_S	F_vS_1/F_aS_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 D 2216
w_a	Force resisting uplift in annular region, N/m (lbf/ft)
w_{AB}	Calculated design uplift load on anchors per unit circumferential length, 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 product 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, knuckles, any permanent attachments and 10% of the roof design snow load, N (lbf)
W_{rs}	Roof load acting on the tank shell including 10% of the roof design snow load, N (lbf)
w_{rs}	Roof load acting on the shell, including 10% of the specified snow load N/m (lbf/ft)
W_s	Total weight of tank shell and appurtenances, N (lbf)
W_T	Total weight of tank shell, roof, framing, knuckles, product, bottom, attachments, appurtenances, participating snow load, if specified, and appurtenances, 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 appendix. 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 maximum considered earthquake ground motion shall be defined as the motion due to an event with a 2% probability of exceedance within a 50-year period. The following definitions apply:

- S_S is the mapped, maximum considered earthquake, 5% damped, spectral response acceleration parameter at short periods (0.2 seconds).

- S_1 is the mapped, maximum considered earthquake, 5% damped, spectral response acceleration parameter at a period of 1 second.
- S_0 is the mapped, maximum considered earthquake, 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.

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 appendix.
- The performance requirements desired by the owner or regulatory body exceed the goal of this appendix.

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 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 response spectra defined in this appendix.

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 Ground Motion

The probabilistic site-specific MCE ground motion shall be taken as that motion represented by a 5% damped acceleration response spectrum having a 2% probability of exceedance in a 50-year period.

E.4.2.3 Deterministic Site-Specific MCE Ground Motion

The deterministic site-specific MCE spectral response acceleration at each period shall be taken as 150% of the largest median 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 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 Ground Motions

The 5% damped site-specific MCE spectral response acceleration at any period, S_a^*, shall be defined as the lesser of the probabilistic MCE 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 appendix, 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 appendix. 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 maximum considered earthquake spectral response accelerations for peak ground acceleration, shall be modified by the appropriate site coefficients, F_a and F_v from Tables E-1 and 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 Maximum Considered Earthquake Spectral Response Accelerations at Short Periods				
	$S_s \le 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.0$	$S_s \ge 1.25$
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 Maximum Considered Earthquake Spectral Response Accelerations at 1 Sec Periods				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
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 (5,000 ft/sec)

B Rock with 760 m/s $< \bar{v}_s \leq$ 1500 m/s (2,500 ft/sec $< \bar{v}_s \leq$ 5,000 ft/sec)

C Very dense soil and soft rock with 360 m/s $< \bar{v}_s \leq$ 760 m/s (1,200 ft/sec $< \bar{v}_s \leq$ 2,500 ft/sec) or with either $\bar{N} > 50$ or $\bar{s}_u > 100$ kPa (2,000 psf)

D Stiff soil with 180 m/s $\leq \bar{v}_s \leq$ 360 m/s (600 ft/sec $\leq \bar{v}_s \leq$ 1,200 ft/sec) or with either $15 \leq \bar{N} \leq 50$ or 50 kPa $\leq \bar{s}_u \leq$ 100 kPa (1,000 psf $\leq \bar{s}_u \leq$ 2,000 psf)

E A soil profile with $\bar{v}_s <$ 180 m/s (600 ft/sec) or with either $\bar{N} < 15$, $\bar{s}_u <$ 50 kPa (1,000 psf), or any profile with more than 3 m (10 ft) of soft clay defined as soil with $PI > 20$, $w \geq 40\%$, 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 Tables E-1 and 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} = the shear wave velocity in m/s (ft/sec),

d_i = 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 = the Standard Penetration Resistance determined in accordance with ASTM D 1586, as directly measured in the field without corrections, and shall not be taken greater than 100 blows/ft.

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

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

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

Use only d_i and N_i for cohesionless soils.

d_s = the total thickness of cohesionless soil layers in the top 30 m (100 ft),

s_{ui} = the undrained shear strength in kPa (psf), determined in accordance with ASTM D 2166 or D 2850, and shall not be taken greater than 240 kPa (5,000 psf).

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

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

d_c = the total thickness ($100 - d_s$) of cohesive soil layers in the top 30 m (100 ft),

PI = the plasticity index, determined in accordance with ASTM D 4318,

w = the moisture content in %, determined in accordance with ASTM D 2216.

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: $s_u < 25$ 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 $\bar{v}_s$, $\bar{N}$, 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. $\bar{N}$ for the top 30 m (100 ft) ($\bar{N}$ method).

c. $\bar{N}$ 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	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
E	(< 180 m/s) (< 600 fps)	< 15	< 50 kPa (< 1,000 psf)
D	180 m/s 360 m/s (600 to 1,200 fps)	15 to 50	50 kPa 100 kPa (1,000 psf – 2,000 psf)
C	360 m/s – 760 m/s (1,200 fps 2,500 fps)	> 50	100 kPa (> 2,000 psf)
B	760 m/s – 1500 m/s (2,500 fps – 5,000 fps)		
A	> 1500 m/s (5,000 fps)		

Note: [a] If the $\bar{s}_u$ method is used and the $\bar{N}_{ch}$ and $\bar{s}_u$ 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 $\overline{v}_s$.

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 appendix 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 appendix are independent of impulsive period of the tank. However, the impulsive period of the tank system may be estimated by Equation E.4.5.1-1.

In SI units:

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

Substituting the SI units specified above: $T_i = 0.128$ sec.

In US Customary units:

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

Substituting the US Customary units specified above: $T_i = 0.128$ sec.

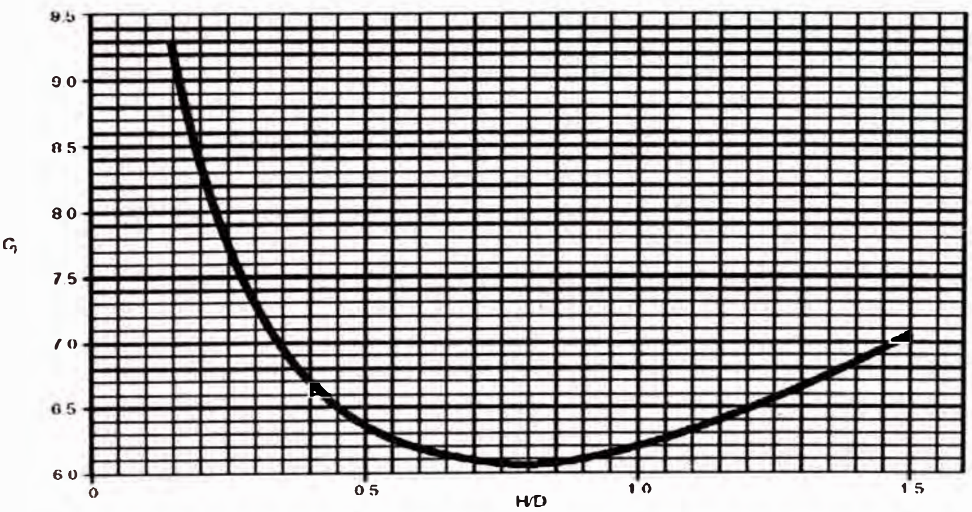


Figure E-1—Coefficient C_i

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:

$$T_c = 1.8K_s\sqrt{D} \tag{E.4.5.2-a}$$

or, in US Customary units:

$$T_c = K_s\sqrt{D} \tag{E.4.5.2-b}$$

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

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 appendix, 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 appendix 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,

$$A_i \geq 0.007 \tag{E.4.6.1-2}$$

and, for seismic site Classes E and F only:

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

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:

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

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;

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

where, S_a^* is the ordinate of the 5% damped, site-specific MCE 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:

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

where, S_a^* is the ordinate of the 5% damped, site-specific MCE 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

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

where

A = lateral acceleration coefficient, %g,

W_{eff} = 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 appendix. 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 appendix, 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.

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

where

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

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

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:

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

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

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

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

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

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:

$$X_i = 0.375H \qquad \text{(E.6.1.2.1-1)}$$

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

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

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

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

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:

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

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.

The maximum vertical seismic acceleration parameter shall be taken as $0.14S_{DS}$ or greater for the ASCE 7 method unless otherwise specified by the Purchaser. Alternatively, the Purchaser may specify the vertical ground motion acceleration parameter, A_v. The total vertical seismic force shall be:

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

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 appendix, 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 $D/H \geq 1.333$:

In SI units:

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

or, in US Customary units:

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

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

In SI units:

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

or, in US Customary units:

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

For $D/H < 1.333$ and $Y \geq 0.75D$:

In SI units:

$$N_i = 2.6 A_i G D^2 \qquad \text{(E.6.1.4-3a)}$$

or, in US Customary units:

$$N_i = 1.39 A_i G D^2 \qquad \text{(E.6.1.4-3b)}$$

For all proportions of D/H:

In SI units:

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

or, in US Customary units:

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

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.

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

When vertical acceleration is specified.

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

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}:

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

Slab Moment, M_s:

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

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 appendix.
- The tanks shall be mechanically anchored to the foundation.
- The value of the base shear and overturning moments for the impulsive mode including the effects of soil-structure interaction shall not be less than 80% of the values determined without consideration of soil-structure interaction.
- 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 appendix. 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 appendix.

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 unanchored tanks.
- 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:

$$w_a = 99t_a\sqrt{F_y HG_e} \leq 201.1\ HDG_e \qquad \text{(E.6.2.1.1-1a)}$$

or, in US Customary units:

$$w_a = 7.9t_a\sqrt{F_y HG_e} \leq 1.28\ HDG_e \qquad \text{(E.6.2.1.1-1b)}$$

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:

$$w_a = 5742\ HG_eL_s \qquad \text{(E.6.2.1.1-2a)}$$

In US Customary units

$$w_a = 36.5\ HG_eL_s \qquad \text{(E.6.2.1.1-2b)}$$

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, $J < 1.54$).
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*

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

where

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

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 Equations E.6.2.1.1.1-1 and 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 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:

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

or, in US Customary units:

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

E.6.2.1.2 Mechanically-Anchored

If the tank configuration is such that the self-anchored requirements can not be met, the tank must be anchored with mechanical devices such as anchor bolts or straps.

When tanks are anchored, the resisting weight of the product shall not be used to reduce the calculated uplift load on the anchors. The anchors shall be sized to provide for at least the following minimum anchorage resistance:

$$w_{AB} = \left(\frac{1.273M_{rw}}{D^2} - w_t(1 - 0.4A_v)\right) \quad \text{(E.6.2.1.2-1)}$$

plus the uplift, in N/m (lbf/ft^2) of shell circumference, due to design internal pressure. See Appendix R for load combinations. If the ratio of operating pressure to design pressure exceeds 0.4, the Purchaser should consider specifying a higher factor on design. Wind loading need not be considered in combination with seismic loading.

The anchor seismic design load, P_{AB}, is defined in Equation E.6.2.1.2-2:

$$P_{AB} = w_{AB}\left(\frac{\pi D}{n_A}\right) \quad \text{(E.6.2.1.2-2)}$$

where, n_A is the number of equally-spaced anchors around the tank circumference. P_{AB} shall be increased to account for unequal spacing.

When mechanical anchorage is required, the anchor embedment or attachment to the foundation, the anchor attachment assembly and the attachment to the shell shall be designed for P_A. The anchor attachment design load, P_A, shall be the lesser of the load equal to the minimum specified yield strength multiplied by the as-built cross-sectional area of the anchor or three times P_{AB}.

The maximum allowable stress for the anchorage parts shall not exceed the following values for anchors designed for the seismic loading alone or in combination with other load cases:

- An allowable tensile stress for anchor bolts and straps equal to 80% of the published minimum yield stress.
- For other parts, 133% of the allowable stress in accordance with 5.10.3.
- The maximum allowable design stress in the shell at the anchor attachment shall be limited to 170 MPa (25,000 lbf/in.^{2}) with no increase for seismic loading. These stresses can be used in conjunction with other loads for seismic loading when the combined loading governs.

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, $J < 0.785$, shall be determined by the formula

In SI units:

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

or, in US Customary units:

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

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:

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

or, in US Customary units:

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

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:

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

or, in US Customary units:

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

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

The maximum longitudinal shell compression stress s_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 ≥44 (SI units) (10^6 US Customary units),

In SI units:

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

or, in US Customary units:

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

In SI units:

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

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

or, in US Customary units:

When GHD^2/t^2 is less than 1×10^6:

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

If the thickness of the bottom shell course calculated to resist the seismic overturning moment is greater than the thickness required for hydrostatic pressure, both excluding any 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.

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

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 ANCHORAGE

Tanks at grade are permitted to be designed without anchorage when they meet the requirements for self-anchored tanks in this appendix.
The following special detailing requirements shall apply to steel tank mechanical anchors in seismic regions where $S_{DS} > 0.05g$.

E.7.1.1 Self-Anchored

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.7.1.2 Mechanically-Anchored

When mechanical-anchorage is required, at least six anchors shall be provided. The spacing between anchors shall not exceed 3 m (10 ft).
When anchor bolts are used, they shall have a minimum diameter of 25 mm (1 in.), excluding any corrosion allowance. Carbon steel anchor straps shall be 6 mm ($^1/_4$ in.) minimum thickness and have a minimum corrosion allowance of 1.5 mm ($^1/_{16}$ in.) on each surface for a distance at least 75 mm (3 in.) but not more than 300 mm (12 in.) above the surface of the concrete.
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 seismic design is required by this appendix. 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.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:

$$\delta_s = 0.5DA_f \text{ (see Note c in Table E-7)} \qquad \text{(E.7.2-1)}$$

For SUG I and II,

When, $T_C \leq 4$

$$A_f = KS_{D1}I\left(\frac{1}{T_C}\right) = 2.5KQF_aS_0I\left(\frac{T_S}{T_C}\right) \qquad \text{(E.7.2-2)}$$

When, $T_C > 4$

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

For SUG III,

When, $T_C \leq T_L$

$$A_f = KS_{D1}\left(\frac{1}{T_C}\right) = 2.5KQF_aS_0\left(\frac{T_S}{T_C}\right) \qquad \text{(E.7.2-4)}$$

When, $T_C > T_L$

$$A_f = KS_{D1}\left(\frac{T_L}{T_C^2}\right) = 2.5KQF_aS_0\left(\frac{T_ST_L}{T_C^2}\right) \qquad \text{(E.7.2-5)}$$

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-7 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:

In SI units:

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

Or, in US Customary units:

$$y_u = \frac{F_y L^2}{83300 t_b} \qquad \text{(E.7.3.1-1b)}$$

where

t_b = calculated annular ring t holdown.

E.7.4 CONNECTIONS

Connections and attachments for anchorage and other lateral force resisting components shall be designed to develop the strength of the anchor (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 an unanchored 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 unanchored 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 appendix (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.).

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

No additional lateral anchorage is required for mechanically-anchored steel tanks designed in accordance with this appendix 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 appendix.

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:

$$V_{max} = \frac{2V}{\pi D} \qquad \text{(E.7.7-1)}$$

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.

E.7.9 SHELL SUPPORT

Self-anchored tanks resting on concrete ringwalls 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 to provide the required uniform support for Items b, c, and 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 bars 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.