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Seismic Design of Liquid-Containing Concrete Structures and Commentary (ACI 350.3-06)

An ACI Standard

Reported by ACI Committee 350

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Seismic Design of Liquid-Containing Concrete Structures and Commentary

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Seismic Design of Liquid-Containing Concrete Structures and Commentary (ACI 350.3-06)

AN ACI STANDARD

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Seismic Design of Liquid-Containing Concrete Structures and Commentary (ACI 350.3-06)

REPORTED BY ACI COMMITTEE 350

This standard prescribes procedures for the seismic analysis and design of liquid-containing concrete structures. These procedures address the loading side of seismic design and are intended to complement ACI 350-06, Section 1.1.8 and Chapter 21.

Keywords: circular tanks; concrete tanks; convective component; earthquake resistance; environmental concrete structures; impulsive component; liquid-containing structures; rectangular tanks; seismic resistance; sloshing; storage tanks.

INTRODUCTION

The following paragraphs highlight the development of this standard and its evolution to the present format:

From the time it embarked on the task of developing an ACI 318-dependent code, ACI Committee 350 decided to expand on and supplement Chapter 21, "Special Provisions for Seismic Design," to provide a set of thorough and comprehensive procedures for the seismic analysis and design of all types of liquid-containing environmental concrete structures. The committee's decision was influenced by the recognition that liquid-containing structures are unique structures whose seismic design is not adequately covered by the leading national codes and standards. A seismic design subcommittee was appointed with the charge to implement the committee's decision.

The seismic subcommittee's work was guided by two main objectives:

1. To produce a self-contained set of procedures that would enable a practicing engineer to perform a full seismic analysis and design of a liquid-containing structure. This meant that these procedures should cover both aspects of seismic design: the "loading side" (namely the determination of the seismic loads based on the mapped maximum considered earthquake spectral response accelerations at short periods (S_s) and 1 second (S_1) obtained from the Seismic Ground Motion maps [Fig. 22-1 through 22-14 of ASCE 7-05, Chapter 22] and the geometry of the structure); and the "resistance side" (the detailed design of the structure in accordance with the provisions of ACI 350, so as to resist those loads safely); and

2. To establish the scope of the new procedures consistent with the overall scope of ACI 350. This required the inclusion of all types of tanks—rectangular, as well as circular; and reinforced concrete, as well as prestressed.

(Note: While there are currently at least two national standards that provide detailed procedures for the seismic analysis and design of liquid-containing structures (ANSI/AWWA 1995a,b), these are limited to circular, prestressed concrete tanks only).

As the loading side of seismic design is outside the scope of ACI 318, Chapter 21, it was decided to maintain this practice in ACI 350 as well. Accordingly, the basic scope, format, and mandatory language of Chapter 21 of ACI 318 were retained with only enough revisions to adapt the chapter to environmental engineering structures. Provisions similar to Section 1.1.8 of ACI 318 are included in ACI 350. This approach offers at least two advantages:

1. It allows ACI 350 to maintain ACI 318's practice of limiting its seismic design provisions to the resistance side only; and

2. It makes it easier to update these seismic provisions so as to keep up with the frequent changes and improvements in the field of seismic hazard analysis and evaluation.

ACI 350.3-06 supersedes 350.3-01 and became effective on July 3, 2006.

The seismic force levels and *R*-factors included in this standard provide results at strength levels, such as those included for seismic design in the 2003 International Building Code (IBC), particularly the applicable connection provisions of 2003 IBC, as referenced in ASCE 7-02. When comparing these provisions with other documents defining seismic forces at allowable stress levels (for example, the 1994 Uniform Building Code [UBC] or ACI 350.3-01), the seismic forces in this standard should be reduced by the applicable factors to derive comparable forces at allowable stress levels.

The user should note the following general design methods used in this standard, which represent some of the key differences relative to traditional methodologies, such as those described in ASCE (1984):

1. Instead of assuming a rigid tank for which the acceleration is equal to the ground acceleration at all locations, this standard assumes amplification of response due to natural frequency of the tank;
2. This standard includes the response modification factor;
3. Rather than combining impulsive and convective modes by algebraic sum, this standard combines these modes by square-root-sum-of-the-squares;
4. This standard includes the effects of vertical acceleration; and
5. This standard includes an effective mass coefficient, applicable to the mass of the walls.

CONTENTS

CHAPTER 1—GENERAL REQUIREMENTS

STANDARD

1.1—Scope

This standard describes procedures for the design of liquid-containing concrete structures subjected to seismic loads. These procedures shall be used in accordance with Chapter 21 of ACI 350-06.

1.2—Notation

A_s = cross-sectional area of base cable, strand, or conventional reinforcement, in.^{2} (mm^2)

b = ratio of vertical to horizontal design acceleration

B = inside dimension (length or width) of a rectangular tank, perpendicular to the direction of the ground motion being investigated, ft (m)

C_c, C_i, and C_t = period-dependent seismic response coefficients defined in 9.4 and 9.5.

C_l, C_w = coefficients for determining the fundamental frequency of the tank-liquid system (refer to Eq. (9-24) and Fig. 9.3.4(b))

C_s = period-dependent seismic coefficient

d, d_{max} = freeboard (sloshing height) measured from the liquid surface at rest, ft (m)

D = inside diameter of circular tank, ft (m)

EBP = excluding base pressure (datum line just above the base of the tank wall)

E_c = modulus of elasticity of concrete, lb/in.^{2} (MPa)

E_s = modulus of elasticity of cable, wire, strand, or conventional reinforcement, lb/in.^{2} (MPa)

F_a = short-period site coefficient (at 0.2 second period) from ASCE 7-05, Table 11.4-1

F_v = long-period site coefficient (at 1.0 second period) from ASCE 7-05, Table 11.4-2

G_p = shear modulus of elastomeric bearing pad, lb/in.^{2} (MPa)

g = acceleration due to gravity [32.17 ft/s^2 (9.807 m/s^2)]

COMMENTARY

R1.1—Scope

This standard is a companion standard to Chapter 21 of the American Concrete Institute, "Code Requirements for Environmental Engineering Concrete Structures and Commentary (ACI 350-06)" (ACI Committee 350 2006).

This standard provides directions to the designer of liquid-containing concrete structures for computing seismic forces that are to be applied to the particular structure. The designer should also consider the effects of seismic forces on components outside the scope of this standard, such as piping, equipment (for example, clarifier mechanisms), and connecting walkways where vertical or horizontal movements between adjoining structures or surrounding backfill could adversely influence the ability of the structure to function properly (National Science Foundation 1981). Moreover, seismic forces applied at the interface of piping or walkways with the structure may also introduce appreciable flexural or shear stresses at these connections.

R1.2—Notation

For C_s, refer to "International Building Code (IBC)" (International Code Council 2003), Section 1617.4.

EBP refers to the hydrodynamic design in which it is necessary to compute the overturning of the wall with respect to the tank floor, excluding base pressure (that is, excluding the pressure on the floor itself). EBP hydrodynamic design is used to determine the need for hold-downs in nonfixed base tanks. EBP is also used in determining the design pressure acting on walls. (For explanation, refer to Housner [1963].)

STANDARD

h_c = height above the base of the wall to the center of gravity of the convective lateral force for the case excluding base pressure (EBP), ft (m)

h_c' = height above the base of the wall to the center of gravity of the convective lateral force for the case including base pressure (IBP), ft (m)

h_i = height above the base of the wall to the center of gravity of the impulsive lateral force for the case excluding base pressure (EBP), ft (m)

h_i' = height above the base of the wall to the center of gravity of the impulsive lateral force for the case excluding base pressure (IBP), ft (m)

h_r = height from the base of the wall to the center of gravity of the tank roof, ft (m)

h_w = height from the base of the wall to the center of gravity of the tank shell, ft (m)

H_L = design depth of stored liquid, ft (m)

H_w = wall height (inside dimension), ft (m)

I = importance factor, from Table 4.1.1(a)

IBP = including base pressure (datum line at the base of the tank including the effects of the tank bottom and supporting structure)

k = flexural stiffness of a unit width of a rectilinear tank wall, lb/ft per foot of wall width (N/m per meter of wall width)

k_a = spring constant of the tank wall support system, lb/ft^2 per foot of wall width (N/m per meter of wall width)

K_a = active coefficient of lateral earth pressure

K_o = coefficient of lateral earth pressure at rest

L = inside dimension of a rectangular tank, parallel to the direction of the ground motion being investigated, ft (m)

L_c = effective length of base cable or strand taken as the sleeve length plus 35 times the strand diameter, in. (mm)

L_p = length of individual elastomeric bearing pads, in. (mm)

m = total mass per unit width of a rectangular wall = $m_i + m_w$, lb-s^2/ft per foot of wall width (kg per meter of wall width)

m_i = impulsive mass of contained liquid per unit width of a rectangular tank wall, lb-s^2/ft per foot of wall width (kg per meter of wall width)

m_w = mass per unit width of a rectangular tank wall, lb-s^2/ft per foot of wall width (kg per meter of wall width)

M_b = bending moment on the entire tank cross section just above the base of the tank wall, ft-lb (kN-m)

M_c = bending moment of the entire tank cross section just above the base of the tank wall (EBP) due to the convective force P_c, ft-lb (kN-m)

COMMENTARY

h = as defined in Section R9.2.4, ft (m)

IBP refers to the hydrodynamic design in which it is necessary to investigate the overturning of the entire structure with respect to the foundation. IBP hydrodynamic design is used to determine the design pressure acting on the tank floor and the underlying foundation. This pressure is transferred directly either to the subgrade or to other supporting structural elements. IBP accounts for moment effects due to dynamic fluid pressures on the bottom of the tank by increasing the effective vertical moment arm to the applied forces. (For explanation, refer to Housner [1963].)

STANDARD

M_c' = overturning moment at the base of the tank, including the tank bottom and supporting structure (IBP), due to the convective force P_c, ft-lb (kN-m)

M_i = bending moment of the entire tank cross section just above the base of tank wall (EBP) due to the impulsive force P_i, ft-lb (kN-m)

M_i' = overturning moment at the base of the tank, including the tank bottom and supporting structure (IBP), due to the impulsive force P_i, ft-lb (kN-m)

M_o = overturning moment at the base of the tank, including the tank bottom and supporting structure (IBP), ft-lb (kN-m)

M_r = bending moment of the entire tank cross section just above the base of the tank wall (EBP) due to the roof inertia force P_r, ft-lb (kN-m)

M_w = bending moment of the entire tank cross section just above the base of the tank wall (EBP) due to the wall inertia force P_w, ft-lb (kN-m)

N_{cy} = in circular tanks, hoop force at liquid level y, due to the convective component of the accelerating liquid, lb per foot of wall height (kN/m)

N_{hy} = in circular tanks, hydrodynamic hoop force at liquid level y, due to the effect of vertical acceleration, lb per foot of wall height (kN/m)

N_{iy} = in circular tanks, hoop force at liquid level y, due to the impulsive component of the accelerating liquid, lb per foot of wall height (kN/m)

N_{wy} = in circular tanks, hoop force at liquid level y, due to the inertia force of the accelerating wall mass, lb per foot of wall height (kN/m)

N_y = in circular tanks, total effective hoop force at liquid level y, lb per foot of wall height (kN/m)

p_{vy} = unit equivalent hydrodynamic pressure due to the effect of vertical acceleration, at liquid level y, above the base of the tank ($p_{vy} = \ddot{u}_v \times q_{hy}$), lb/ft^2 (kPa)

P_c = total lateral convective force associated with W_c, lb (kN)

P_{cy} = lateral convective force due to W_c, per unit height of the tank wall, occurring at liquid level y, lb per foot of wall height (kN/m)

P_{eg} = lateral force on the buried portion of a tank wall due to the dynamic earth and groundwater pressures, lb (kN)

COMMENTARY

p_{cy} = unit lateral dynamic convective pressure distributed horizontally at liquid level y, lb/ft^2 (kPa)

p_{iy} = unit lateral dynamic impulsive pressure distributed horizontally at liquid level y, lb/ft^2 (kPa)

p_{wy} = unit lateral inertia force due to wall dead weight, distributed horizontally at liquid level y, lb/ft^2 (kPa)

STANDARD

P_h = total hydrostatic force occurring on length B of a rectangular tank or diameter D of a circular tank, lb (kN)

P_{hy} = lateral hydrostatic force per unit height of the tank wall, occurring at liquid level y, lb per foot of wall height (kN/m)

P_i = total lateral impulsive force associated with W_i, lb (kN)

P_{iy} = lateral impulsive force due to W_i, per unit height of the tank wall, occurring at liquid level y, lb per foot of wall height (kN/m)

P_r = lateral inertia force of the accelerating roof W_r, lb (kN)

P_w = lateral inertia force of the accelerating wall W_w, lb (kN)

P'_w = in a rectangular tank, lateral inertia force of one accelerating wall (W'_w), perpendicular to the direction of the earthquake force, lb (kN)

P_{wy} = lateral inertia force due to W_w, per unit height of the tank wall, occurring at level y above the tank base, lb per foot of wall height (kN/m)

P_y = combined horizontal force (due to the impulsive and convective components of the accelerating liquid; the wall's inertia, and the hydrodynamic pressure due to the vertical acceleration) at a height y above the tank base, lb per foot of wall height (kN/m)

q_{hy} = unit hydrostatic pressure at liquid level y above the tank base [$q_{hy} = \gamma_L (H_L - y)$], lb/ft^2 (kPa)

r = inside radius of circular tank, ft (m)

R = response modification factor, a numerical coefficient representing the combined effect of the structure's ductility, energy-dissipating capacity, and structural redundancy (R_c for the convective component of the accelerating liquid; R_i for the impulsive component) from Table 4.1.1(b)

COMMENTARY

For a schematic representation of P_h, refer to Fig. R5.3.1(a) and (b)

q, q_{max} = unit shear force in circular tanks, lb/ft (kN/m)

Q = total membrane (tangential) shear force at the base of a circular tank, lb (kN)

Q_{hy} = in circular tanks, hydrostatic hoop force at liquid level y ($Q_{hy} = q_{hy} \times R$), lb per foot of wall height (kN/m)

S_0 = effective peak ground acceleration (at T = 0) related to the maximum considered earthquake; expressed as a fraction of the acceleration due to gravity g from a site-specific spectrum. (S_0 is equivalent to a PGA having a 2% probability of exceedance in 50 years, as given in the U.S. Geological Survey (USGS) database at website (http://eqhazmaps.usgs.gov)

STANDARD

S_1 = mapped maximum considered earthquake 5% damped spectral response acceleration; parameter at a period of 1 second, expressed as a fraction of the acceleration due to gravity g, from ASCE 7-05, Fig. 22-1 through 22-14

S_{aM} = maximum considered earthquake spectral response acceleration, 5% damped, at period T_i or T_v, taken from a site-specific acceleration response spectrum

S_c = center-to-center spacing between individual base cable loops, in. (mm)

S_{cM} = maximum considered earthquake spectral response acceleration, 0.5% damped, at period T_c, taken from a site-specific acceleration response spectrum

S_{D1} = design spectral response acceleration, 5% damped, at a period of 1 second, as defined in 9.4.1, expressed as a fraction of the acceleration due to gravity g

S_{DS} = design spectral response acceleration, 5% damped, at short periods, as defined in 9.4.1, expressed as a fraction of the acceleration due to gravity g

S_p = center-to-center spacing of elastomeric bearing pads, in. (mm)

S_s = mapped maximum considered earthquake 5% damped spectral response acceleration parameter at short periods, expressed as a fraction of the acceleration due to gravity g, from ASCE 7-05, Fig. 22-1 through 22-14

t_p = thickness of elastomeric bearing pads, in. (mm)

t_w = average wall thickness, in. (mm)

T_c = natural period of the first (convective) mode of sloshing, s

T_i = fundamental period of oscillation of the tank (plus the impulsive component of the contents), s

T_S = S_{D1}/S_{DS}

COMMENTARY

S_a = generalized design spectral response acceleration corresponding to a given natural period T, expressed as a fraction of the acceleration due to gravity g

S_D = spectral displacement, ft (m)

T_S: In Appendix B,

$$T_S = \frac{C_v}{2.5C_a} = 0.40\frac{C_v}{C_a}$$

where C_a and C_v are defined in Appendix B.

STANDARD

T_v = natural period of vibration of vertical liquid motion, s

$\ddot{u}_v$ = effective spectral acceleration from an inelastic vertical response spectrum, as defined by Eq. (4-15), that is derived by scaling from an elastic horizontal response spectrum, expressed as a fraction of the acceleration due to gravity g

V = total horizontal base shear, lb (kN)

w_p = width of elastomeric bearing pad, in. (mm)

W_c = equivalent weight of the convective component of the stored liquid, lb (kN)

W_e = effective dynamic weight of the tank structure (walls and roof) [$W_e = (\varepsilon W_w + W_r)$], lb (kN)

W_i = equivalent weight of the impulsive component of the stored liquid, lb (kN)

W_L = total equivalent weight of the stored liquid, lb (kN)

W_r = equivalent weight of the tank roof, plus superimposed load, plus applicable portion of snow load considered as dead load, lb (kN)

W_w = equivalent weight of the tank wall (shell), lb (kN)

W_w' = in a rectangular tank, the equivalent weight of one wall perpendicular to the direction of the earthquake force, lb (kN)

y = liquid level at which the wall is being investigated (measured from tank base), ft (m)

α = angle of base cable or strand with horizontal, degree

β = percent of critical damping

γ_c = density of concrete, [150 lb/ft^3 (23.56 kN/m^3) for standard-weight concrete]

γ_L = density of contained liquid, lb/ft^3 (kN/m^3)

γ_w = density of water, 62.43 lb/ft^3 (9.807 kN/m^3)

ε = effective mass coefficient (ratio of equivalent dynamic mass of the tank shell to its actual total mass), Eq. (9-44) and (9-45)

θ = polar coordinate angle, degree

λ = coefficient as defined in 9.2.4 and 9.3.4

σ_y = membrane (hoop) stress in wall of circular tank at liquid level y, lb/in.^{2} (MPa)

ω_c = circular frequency of oscillation of the first (convective) mode of sloshing, radian/s

ω_i = circular frequency of the impulsive mode of vibration, radian/s

COMMENTARY

η_c, η_i = coefficients as defined in Section R4.2

For θ, refer to Fig. R5.2.1 and R5.2.2.

CHAPTER 2—TYPES OF LIQUID-CONTAINING STRUCTURES

STANDARD

2.1—Ground-supported structures

Structures in this category include rectangular and circular liquid-containing concrete structures, on-grade and below grade.

2.1.1—Ground-supported liquid-containing structures are classified according to this section on the basis of the following characteristics:

- General configuration (rectangular or circular);
- Wall-base joint type (fixed, hinged, or flexible base); and
- Method of construction (reinforced or prestressed concrete).

Type 1—Rectangular tanks
Type 1.1—Fixed base
Type 1.2—Hinged base

Type 2—Circular tanks
Type 2.1—Fixed base
2.1(1)—Reinforced concrete
2.1(2)—Prestressed concrete
Type 2.2—Hinged base
2.2(1)—Reinforced concrete
2.2(2)—Prestressed concrete
Type 2.3—Flexible base (prestressed only)
2.3(1)—Anchored
2.3(2)—Unanchored, contained
2.3(3)—Unanchored, uncontained

2.2—Pedestal-mounted structures

Structures in this category include liquid-containing structures mounted on cantilever-type pedestals.

COMMENTARY

R2.1—Ground-supported structures

For basic configurations of ground-supported, liquid-containing structures, refer to Fig. R2.1.

R2.1.1—The classifications of Section 2.1.1 are based on the wall-to-footing connection details as illustrated in Fig. R2.1.1.

The tank floor and floor support system should be designed for the seismic forces transmitted therein. With any one of the tank types covered under this standard, the floor may be a membrane-type slab, a raft foundation, or a structural slab supported on piles. This standard, however, does not cover the determination of seismic forces on the piles themselves.

The tank roof may be a free-span dome or column-supported flat slab, or the tank may be open-topped.

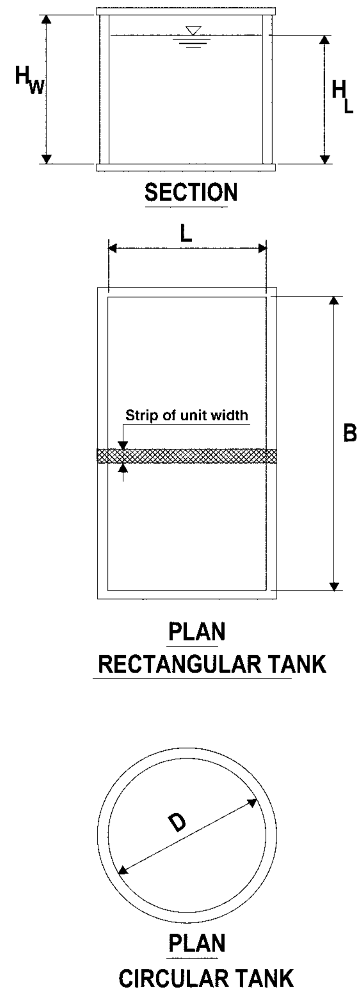


Fig. R2.1—Typical tank configurations (adapted from ASCE [1984]).

COMMENTARY

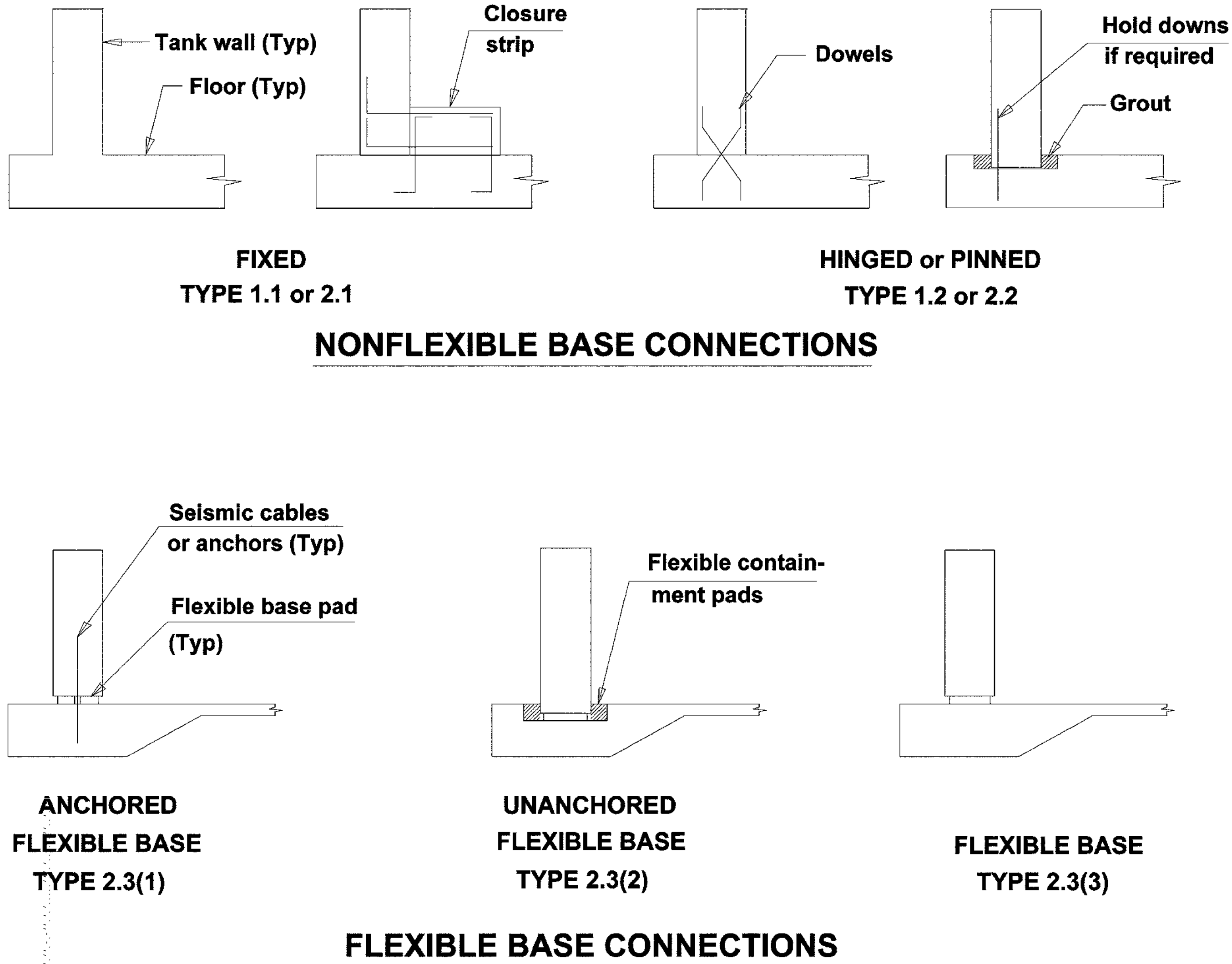


Fig. R2.1.1—Types of ground-supported, liquid-containing structures classified on the basis of their wall-to-footing connection details (base waterstops not shown).

CHAPTER 3—GENERAL CRITERIA FOR ANALYSIS AND DESIGN

STANDARD

3.1—Dynamic characteristics

The dynamic characteristics of liquid-containing structures shall be derived in accordance with either Chapter 9 or a more rigorous dynamic analysis that accounts for the interaction between the structure and the contained liquid.

3.2—Design loads

The loads generated by the design earthquake shall be computed in accordance with Chapter 4.

3.3—Design requirements

3.3.1—The walls, floors, and roof of liquid-containing structures shall be designed to withstand the effects of both the design horizontal acceleration and the design vertical acceleration combined with the effects of the applicable design static loads.

3.3.2—With regards to the horizontal acceleration, the design shall take into account the effects of the transfer of the total base shear between the wall and the footing and between the wall and the roof, and the dynamic pressure acting on the wall above the base.

3.3.3—Effects of maximum horizontal and vertical acceleration shall be combined by the square-root-sum-of-the-squares method.

COMMENTARY

R3.1—Dynamic characteristics

For an outline of the general steps involved in the interaction between the structure and the contained liquid, refer to Appendix A.

Notes

CHAPTER 4—EARTHQUAKE DESIGN LOADS

STANDARD

4.1—Earthquake pressures above base

The walls of liquid-containing structures shall be designed for the following dynamic forces in addition to the static pressures in accordance with Section 5.3.1:

(a) Inertia forces P_w and P_r;
(b) Hydrodynamic impulsive force P_i from the contained liquid;
(c) Hydrodynamic convective force P_c from the contained liquid;
(d) Dynamic earth pressure from saturated and unsaturated soils against the buried portion of the wall; and
(e) The effects of vertical acceleration.

COMMENTARY

R4.1—Earthquake pressures above base

The general equation for the total base shear normally encountered in the earthquake-design sections of governing building codes $V = C_s W$ is modified in Eq. (4-1) through (4-4) by replacing the term W with the four effective weights: the effective weight of the tank wall εW_w and roof W_r; the impulsive component of the liquid weight W_i and the convective component W_c; and the term C_s with C_i, C_c, or C_v as appropriate. Because the impulsive and convective components are not in phase with each other, practice is to combine them using the square-root sum-of-the-squares method (Eq. (4-5)) (New Zealand Standard [NZS] 1986; ASCE 1981; ANSI/AWWA 1995a).

A more detailed discussion of the impulsive and convective components W_i and W_c is contained in Section R9.1.

The imposed ground motion is represented by an elastic response spectrum that is either derived from an actual earthquake record for the site, or is constructed by analogy to sites with known soil and seismic characteristics. The profile of the response spectrum is defined by S_a, which is a function of the period of vibration and the mapped accelerations S_S and S_1 as described in Section R9.4.

Factor I provides a means for the engineer to increase the factor of safety for the categories of structures described in Table 4.1.1(a). Engineering judgment may require a factor I greater than tabulated in Table 4.1.1(a) where it is necessary to reduce further the potential level of damage or account for the possibility of an earthquake greater than the design earthquake.

The response modification factors R_c and R_i reduce the elastic response spectrum to account for the structure's ductility, energy-dissipating properties, and redundancy (ACI 350-06, Section R21.2.1).

Explanations of the impulsive and convective pressures P_i and P_c are contained in Section R9.1 and Housner (1963).

STANDARD

4.1.1—Dynamic lateral forces

The dynamic lateral forces above the base shall be determined as

$$P_w = C_i I \left[\frac{\varepsilon W_w}{R_i}\right] \quad (4\text{-}1)$$

$$P'_w = C_i I \left[\frac{\varepsilon W'_w}{R_i}\right] \quad (4\text{-}1a)$$

$$P_r = C_i I \left[\frac{W_r}{R_i}\right] \quad (4\text{-}2)$$

$$P_i = C_i I \left[\frac{W_i}{R_i}\right] \quad (4\text{-}3)$$

$$P_c = C_c I \left[\frac{W_c}{R_c}\right] \quad (4\text{-}4)$$

where C_i and C_c are the seismic response coefficients determined in accordance with Sections 9.4 and 9.5; I is the importance factor defined in Table 4.1.1(a); W_w and W_r are the weights of the cylindrical tank wall (shell) and tank roof, respectively, and W'_w is the weight of one wall in a rectangular tank, perpendicular to the direction of the ground motion being investigated; ε is a factor defined in Section 1.2 and determined in accordance with Section 9.6; W_i and W_c are the impulsive and convective components of the stored liquid, respectively, as defined in Section 1.2 and determined in accordance with Sections 9.2.1 and 9.3.1; and R_i and R_c are the response modification factors defined in Section 1.2 and determined in accordance with Table 4.1.1(b).

Where applicable, the lateral forces due to the dynamic earth and groundwater pressures against the buried portion of the walls shall be computed in accordance with the provisions of Chapter 8.

4.1.2—Total base shear

The base shear due to seismic forces applied at the bottom of the tank wall shall be determined by

$$V = \sqrt{(P_i + P_w + P_r)^2 + P_c^2 + P_{eg}^2} \quad (4\text{-}5)$$

Where applicable, the lateral forces due to dynamic earth and groundwater pressures against the buried

COMMENTARY

R4.1.1—Dynamic lateral forces

A model representation of W_i and W_c is shown in Fig. R9.1.

Energy Method: An energy method of dynamic analysis may be used instead of the base-shear approach of Section 4.1 for sizing earthquake cables and base pad for flexible base joints (Housner 1963; John A. Blume & Associates 1958; Medearis and Young 1964; Uang and Bertero 1988; Bertero 1995; Scarlat 1997).

STANDARD

portion of the walls shall be included in the determination of the total base shear ***V***.

4.1.3—Moments at base, general equation

The moments due to seismic forces at the base of the tank shall be determined by Eq. (4-10) and (4-13).

Bending moment on the entire tank cross section just above the base of the tank wall (EBP)

$$M_w = P_w h_w \tag{4-6}$$

$$M_r = P_r h_r \tag{4-7}$$

$$M_i = P_i h_i \tag{4-8}$$

$$M_c = P_c h_c \tag{4-9}$$

$$M_b = \sqrt{(M_i + M_w + M_r)^2 + M_c^2} \tag{4-10}$$

Overturning moment at the base of the tank, including the tank bottom and supporting structure (IBP)

$$M_i' = P_i h_i' \tag{4-11}$$

$$M_c' = P_c h_c' \tag{4-12}$$

$$M_o = \sqrt{(M_i' + M_w + M_r)^2 + M_c'^2} \tag{4-13}$$

Where applicable, the effect of dynamic earth and groundwater pressures against the buried portion of the walls shall be included in the determination of the moments at the base of the tank.

4.1.4—Vertical acceleration

4.1.4.1 The tank shall be designed for the effects of vertical acceleration. In the absence of a site-specific response spectrum, the ratio ***b*** of the vertical-to-horizontal acceleration shall not be less than 2/3

4.1.4.2 The hydrostatic load q_{hy} from the tank contents, at level ***y*** above the base, shall be multiplied by the spectral acceleration $\ddot{u}_v$ to account for the effect of the vertical acceleration. The resulting hydrodynamic pressure p_{vy} shall be computed as

$$p_{vy} = \ddot{u}_v q_{hy} \tag{4-14}$$

COMMENTARY

R4.1.4—Vertical acceleration

The effective fluid pressure will be increased or decreased due to the effects of vertical acceleration. Similar changes in effective weight of the concrete structure may also be considered.

IBC (2003), Section 1617.1.1, and Building Seismic Safety Council (2000), Section 5.2.7, use a factor of $0.2S_{DS}$ to account for the effects of vertical ground acceleration in the definition of seismic effects.

STANDARD

where

$$\ddot{u}_v = C_t I \left[\frac{b}{R_i}\right] \geq 0.2 S_{DS} \qquad (4\text{-}15)$$

where C_t is the seismic response coefficient determined in accordance with Sections 9.4 and 9.5.

4.2—Application of site-specific response spectra

4.2.1—General

Where site-specific procedures are used, the maximum considered earthquake spectral response acceleration shall be taken as the lesser of the probabilistic maximum earthquake spectral response acceleration as defined in Section 4.2.2 and the deterministic maximum spectral response acceleration as defined in Section 4.2.3.

4.2.2—Probabilistic maximum considered earthquake

The probabilistic maximum considered earthquake spectral response acceleration shall be taken as the spectral response acceleration represented by a 5% damped acceleration response spectrum having a 2% probability of exceedance in a 50-year period.

COMMENTARY

R4.2—Application of site-specific response spectra

R4.2.1—General

In locations with $S_s \geq 1.5$ or $S_1 \geq 0.60$ and sites with weak soil conditions, site-specific response spectra are normally used.

R4.2.2—Probabilistic maximum considered earthquake

For probabilistic ground motions, a 2% probability of exceedance in a 50-year period is equivalent to a recurrence interval of approximately 2500 years.

When the available site-specific response spectrum is for a damping ratio β other than 5% of critical, the period-dependent spectral acceleration S_{aM} given by such site-specific spectrum should be modified by the factor η_i to account for the influence of damping on the spectral amplification as follows (Newmark and Hall 1982)

For 0 seconds < (T_i or T_v) < T_s

$$\eta_i = \frac{2.706}{4.38 - 1.04 \ln \beta}$$

For T_s < (T_i or T_v) < 4.0 seconds

$$\eta_i = \frac{2.302}{3.38 - 0.67 \ln \beta}$$

For β = 5%, η_i = 1.0

When the available site-specific response spectrum is for a damping ratio β other than 0.5% of critical, the period-dependent spectral acceleration S_{cM} given by that spectrum may be modified by the ratio η_c to account for the influence of damping on the spectral amplification as follows

$$\eta_c = \frac{3.043}{2.73 - 0.45 \ln \beta}$$

STANDARD

COMMENTARY

For β = 0.5%, η_c = 1.0

For site-specific response spectra drawn on a tripartite logarithmic plot, the design spectral acceleration S_{cM} can also be derived by using the relationship

$$S_{cM} = \eta_c \frac{S_D}{g}\left(\frac{2\pi}{T_c}\right)^2 = \eta_c \frac{1.226 S_D}{T_c^2}$$

where S_D is the spectral displacement corresponding to T_c obtained directly from the site-specific spectrum in the range $T_c > 1.6/T_s$.

4.2.3—Deterministic maximum considered earthquake

The deterministic maximum considered earthquake 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 on all known active faults within the region. The deterministic value of the spectral response acceleration shall not be taken lower than $0.6F_v/T$ except that the lower limit of the spectral response acceleration shall not exceed $1.5F_a$. The site coefficients F_a and F_v shall be obtained from ASCE 7-05, Tables 11.4-1 and 11.4-2, respectively.

R4.2.3—Deterministic maximum considered earthquake

For deterministic ground motions, the magnitude of a characteristic earthquake on a given fault should be the best estimate of the maximum magnitude capable for that fault, and should not be less than the largest magnitude that has occurred historically on the fault.

4.2.4 The maximum considered earthquake spectral response accelerations S_{aM} and S_{cM} shall be determined in accordance with Section 9.5 using the site-specific acceleration response spectrum as defined in Section 4.2.1.

STANDARD

Table 4.1.1(a)—Importance factor *I*

	Tank use	Factor *I*
III	Tanks containing hazardous materials*	1.5
II	Tanks that are intended to remain usable for emergency purposes after an earthquake, or tanks that are part of lifeline systems	1.25
I	Tanks not listed in Categories II or III	1.0

*In some cases, for tanks containing hazardous materials, engineering judgment may require a factor I > 1.5.

Table 4.1.1(b)—Response modification factor *R*

Type of structure	R_i: On or above grade	R_i: Buried*	R_c
Anchored, flexible-base tanks	3.25†	3.25†	1.0
Fixed or hinged-base tanks	2.0	3.0	1.0
Unanchored, contained, or uncontained tanks‡	1.5	2.0	1.0
Pedestal-mounted tanks	2.0	—	1.0

*Buried tank is defined as a tank whose maximum water surface at rest is at or below ground level. For partially buried tanks, the R_i value may be linearly interpolated between that shown for tanks on grade and for buried tanks.

†R_i = 3.25 is the maximum R_i value permitted to be used for any liquid-containing concrete structure.

‡Unanchored, uncontained tanks shall not be built in locations where $S_{DS} \geq 0.75$.

CHAPTER 5—EARTHQUAKE LOAD DISTRIBUTION

STANDARD

5.1—General

In the absence of a more rigorous analysis that takes into account the complex vertical and horizontal variations in hydrodynamic pressures, liquid-containing structures shall be designed for the following dynamic shear and pressure distributions in addition to the static load distributions:

5.2—Shear transfer

5.2.1—Rectangular tanks

The wall-to-floor, wall-to-wall, and wall-to-roof joints of rectangular tanks shall be designed for the earthquake shear forces on the basis of the following shear-transfer mechanism:

- Walls perpendicular to the direction of the ground motion being investigated shall be analyzed as slabs subjected to the horizontal pressures computed in Section 5.3. The shears along the bottom and side joints and the top joint in case of a roof-covered tank shall correspond to the slab reactions; and
- Walls parallel to the direction of the ground motion being investigated shall be analyzed as shearwalls subjected to the in-plane forces computed in Section 5.3.

5.2.2—Circular tanks

The wall-to-footing and wall-to-roof joints shall be designed for the earthquake shear forces.

COMMENTARY

R5.2— Shear transfer (NZS 1986)

The horizontal earthquake force $\boldsymbol{V}$ generates shear forces between the wall and footing and the wall and roof.

R5.2.1—Rectangular tanks

Typically, the distribution of forces and wall reactions in rectangular tank walls will be similar to that shown in Fig. R5.2.1.

R5.2.2—Circular tanks

In fixed- and hinged-base circular tanks (Types 2.1 and 2.2), the earthquake base shear is transmitted partially by membrane (tangential) shear and the rest by radial shear that causes vertical bending. For a tank with a height-to-diameter ratio of 1:4 ($\boldsymbol{D/H_L} = \mathbf{4.0}$), approximately 20% of the earthquake shear force is transmitted by the radial base reaction to vertical bending. The remaining 80% is transmitted by tangential shear transfer $\boldsymbol{Q}$. To transmit this tangential shear $\boldsymbol{Q}$, a distributed shear force $\boldsymbol{q}$ is required at the wall/footing interface, where

$$q = \frac{Q}{\pi r}\sin\theta$$

The distribution is illustrated in Fig. R5.2.2.

The maximum tangential shear occurs at a point on the tank wall oriented 90 degrees to the design earthquake direction being evaluated and is given by

STANDARD

COMMENTARY

$$q_{max} = \frac{Q}{\pi r} = \frac{0.8V}{\pi r}$$

The radial shear is created by the flexural response of the wall near the base, and is therefore proportional to the hydrodynamic forces shown in Fig. R5.2.1. The radial shear attains its maximum value at points on the tank wall oriented zero and 180 degrees to the ground motion and should be determined using cylindrical shell theory and the tank dimensions. The design of the wall-footing interface should take the radial shear into account.

In general, the wall-footing interface should have reinforcement designed to transmit these shears through the joint. Alternatively, the wall may be located in a preformed slot in the ring beam footing.

In anchored, flexible-base, circular tanks (Type 2.3(1)) it is assumed that the entire base shear is transmitted by membrane (tangential) shear with only insignificant vertical bending.

$$Q = 1.0V$$

$$q_{max} = \frac{Q}{\pi r} = \frac{V}{\pi r}$$

In tank Types 2.3(2) and 2.3(3) it is assumed that the base shear is transmitted by friction only. If friction between the wall base and the footing, or between the wall base and the bearing pads, is insufficient to resist the earthquake shear, some form of mechanical restraint such as dowels, galvanized steel cables, or preformed slots may be required.

Failure to provide a means for shear transfer around the circumference may result in sliding of the wall.

When using preformed slots, vertical bending moments induced in the wall by shear should be considered.

The roof-to-wall joint is subject to earthquake shear from the horizontal acceleration of the roof. Where dowels are provided to transfer this shear, the distribution will be the same as shown in Fig. R5.2.2 with maximum shear given by

$$q_{max} = \frac{0.8P_r}{\pi r}$$

where P_r is the force from the horizontal acceleration of the roof.

For tanks with roof overhangs, the concrete lip can be designed to withstand the earthquake force. Because the roof is free to slide on top of the wall, the shear transfer will take place over that portion of the circumference where the lip overhang comes into contact with the wall. Typically, the distribution of forces and wall reactions in circular tanks

STANDARD

COMMENTARY

will be similar to that shown in Fig. R5.2.1, but reacting on only half of the circumference. The maximum reaction force will be given by

$$q_{max} = \frac{2.0P_r}{\pi r}$$

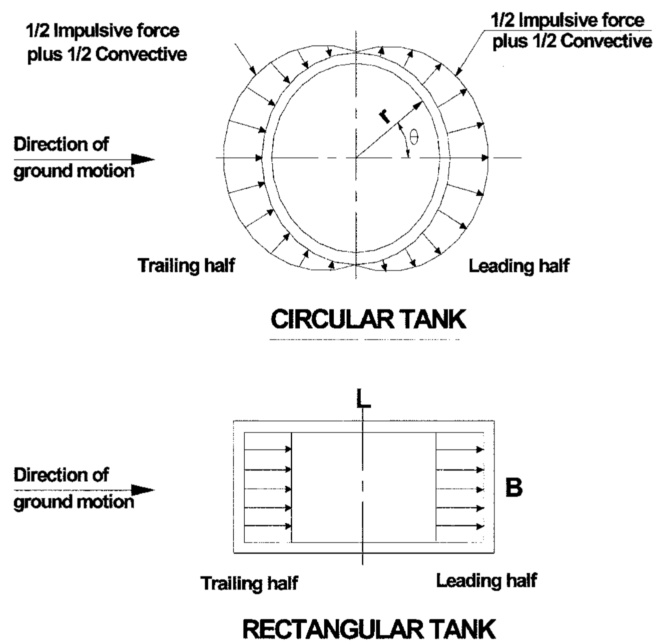


Fig. R5.2.1—Hydrodynamic pressure distribution in tank walls (adapted from Housner [1963] and NZS [1986]).

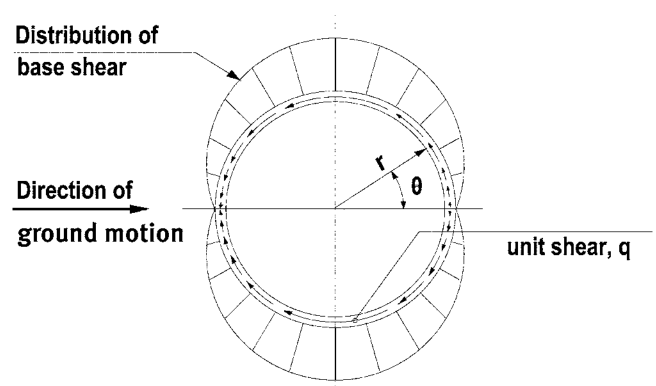


Fig. R5.2.2—Membrane shear transfer at the base of circular tanks (adapted from NZS [1986]).

STANDARD

5.3—Dynamic force distribution above base

5.3.1—Rectangular tanks

Walls perpendicular to the ground motion being investigated and in the leading half of the tank shall be loaded perpendicular to their plane (dimension B) by the wall's own inertia force P'_W, one-half the impulsive force P_i, and one-half the convective force P_c.

Walls perpendicular to the ground motion being investigated and in the trailing half of the tank shall be loaded perpendicular to their plane (dimension B) by the wall's own inertia force P'_W, one-half the impulsive force P_i; one-half the convective force P_c, and the dynamic earth and groundwater pressure against the buried portion of the wall.

Walls parallel to the direction of the ground motion being investigated shall be loaded in their plane (dimension L) by: (a) the wall's own in-plane inertia force P'_W and the in-plane forces corresponding to the edge reactions from the abutting wall(s).

Superimposed on these lateral unbalanced forces shall be the lateral hydrodynamic force resulting from the hydrodynamic pressure due to the effect of vertical acceleration p_{vy} acting on each wall.

5.3.2—Combining dynamic forces for rectangular tanks

The hydrodynamic force at any given height y from the base shall be determined by

$$P_y = \sqrt{(P_{iy} + P_{wy})^2 + P_{cy}^2 + (p_{vy}B)^2} \qquad (5\text{-}1)$$

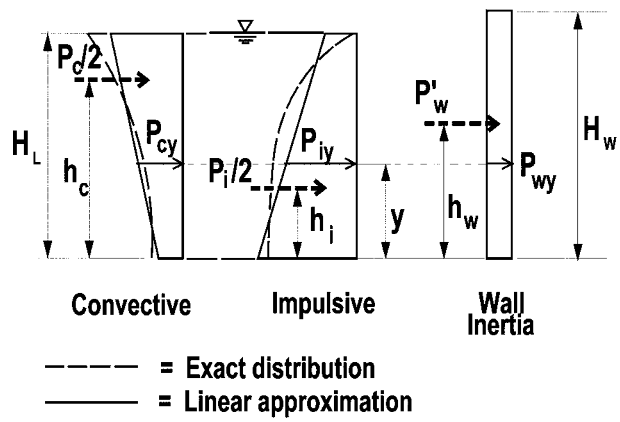
Where applicable, the effect of the dynamic earth and groundwater pressures against the buried portion of the walls shall be included.

COMMENTARY

R5.3—Dynamic force distribution above base

R5.3.1—Rectangular tanks

The vertical distribution, per foot of wall height, of the dynamic forces acting perpendicular to the plane of the wall may be assumed as shown below (adapted from NZS [1986], Section 2.2.9.5), and Fig. R5.3.1(b).



$$P_{wy} = (C_i I/R_i) \times [\varepsilon(\gamma_c B t_w)]/12$$

$$[P_{wy} = (C_i I/R_i) \times [\varepsilon(\gamma_c B t_w)] \text{ in SI}]$$

$$P_{iy} = \frac{\frac{P_i}{2}\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{H_L^2}$$

$$P_{cy} = \frac{\frac{P_c}{2}\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{H_L^2}$$

Fig. R5.3.1(a)—Vertical force distribution: rectangular tanks.

The horizontal distribution of the dynamic pressures across the wall width B is

$$p_{wy} = \frac{P_{wy}}{B} \qquad p_{cy} = \frac{P_{cy}}{B}$$

$$p_{iy} = \frac{P_{iy}}{B} \qquad p_{vy} = \ddot{u}_v q_{hy}$$

The dynamic force on the leading half of the tank will be additive to the hydrostatic force on the wall, and the dynamic force on the trailing half of the tank will reduce the effects of the hydrodynamic force on the wall.

COMMENTARY

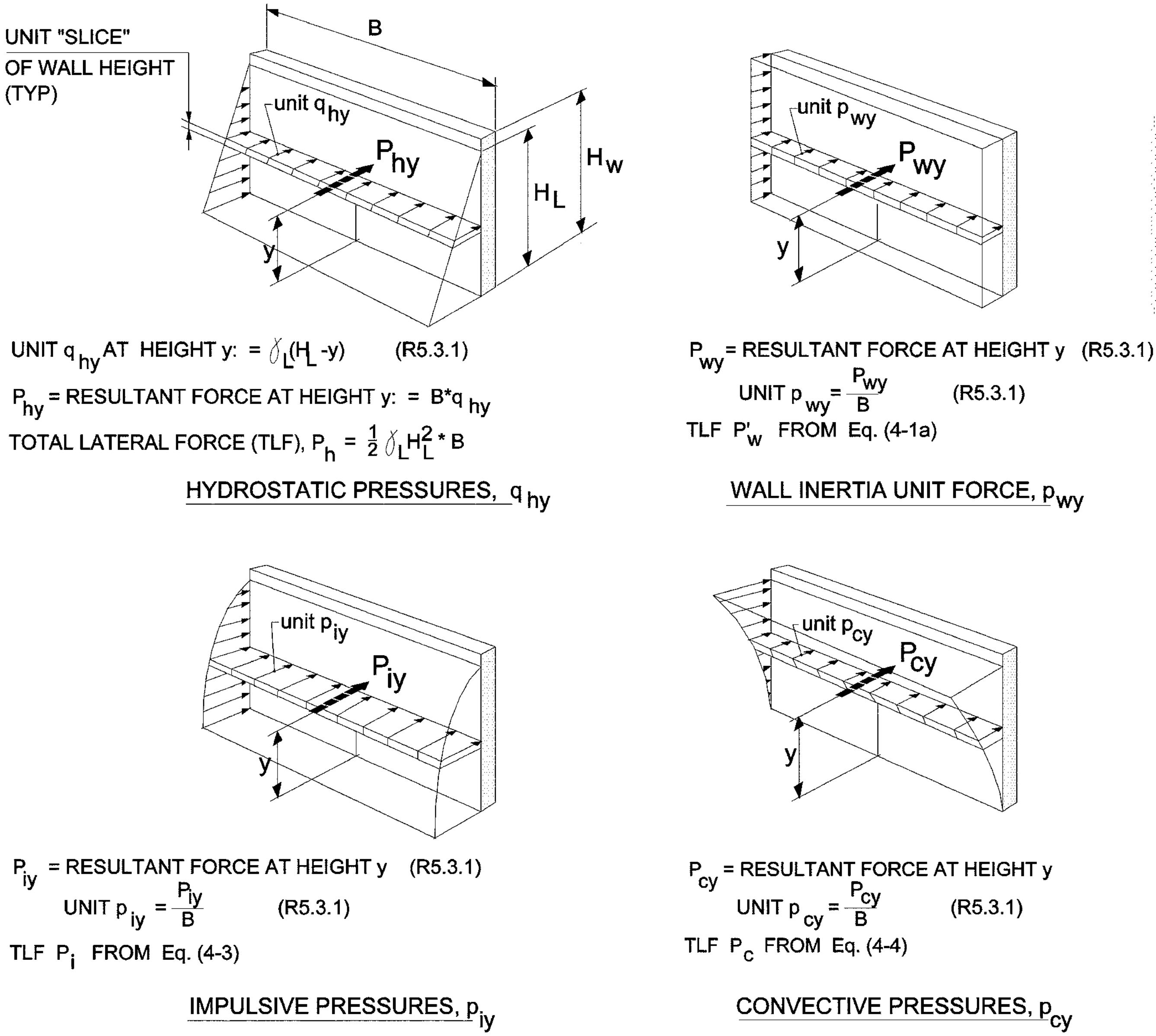


Fig. R5.3.1(b)—Distribution of hydrostatic and hydrodynamic pressures and inertia forces on the wall of a rectangular liquid-containing structure (adapted from Haroun [1984]). (For circular tanks, the vertical distribution of the impulsive and convective forces is identical to that shown above for rectangular tanks, while the horizontal distribution varies along the tank circumference as shown in Fig. R5.2.1.)

STANDARD

5.3.3—Circular tanks

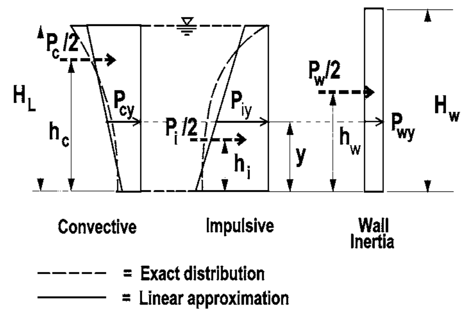
The cylindrical walls of circular tanks shall be loaded by the wall's own inertia force distributed uniformly around the entire circumference; one-half the impulsive force P_i applied symmetrically about $\theta = 0$ degrees and acting outward on one half of the wall's circumference, and one-half P_i symmetrically about $\theta = 180$ degrees and acting inward on the opposite half of the wall's circumference; one-half the convective force P_c acting on one-half of the wall's circumference symmetrically about $\theta = 0$ degrees and one-half P_c symmetrically about $\theta = 180$ degrees and acting inward on the opposite half of the wall's circumference; and the dynamic earth and groundwater pressure against the trailing half of the buried portion of the wall.

Superimposed on these lateral unbalanced forces shall be the axisymmetric lateral hydrodynamic force resulting from the hydrodynamic pressure p_{vy} acting on the tank wall.

COMMENTARY

R5.3.3—Circular tanks

The vertical distribution, per foot of wall height, of the dynamic forces acting on one half of the wall may be assumed as shown below and in Fig. R5.3.3 and Fig. R5.2.1.



$$P_{wy} = \frac{P_w}{2H_w}$$ (for constant-thickness wall. For tapered wall, modify accordingly)

$$P_{iy} = \frac{\frac{P_i}{2}\left[4H_L - 6h_i - (6H_L - 12h_i)\left(\frac{y}{H_L}\right)\right]}{H_L^2}$$

$$P_{cy} = \frac{\frac{Pc}{2}\left[4H_L - 6h_c - (6H_L - 12h_c)\left(\frac{y}{H_L}\right)\right]}{H_L^2}$$

Fig. R5.3.3—Vertical force distribution: circular tanks.

The horizontal distribution of the dynamic pressure across the tank diameter D may be assumed as follows

$$p_{wy} = \frac{P_{wy}}{\pi r} \qquad p_{cy} = \frac{16P_{cy}}{9\pi r}\cos\theta$$

$$p_{iy} = \frac{2P_{iy}}{\pi r}\cos\theta \qquad p_{vy} = \ddot{u}_v q_{hy}$$

CHAPTER 6—STRESSES

STANDARD

COMMENTARY

R6—General

In calculating the vertical bending moments in the walls of rectangular and circular tanks, the boundary conditions at the wall-to-base and wall-to-roof joints should be properly accounted for. Typical earthquake force distributions in walls of rectangular and circular tanks are presented in R5.3.1 and R5.3.3, respectively.

6.1—Rectangular tanks

The vertical and horizontal bending stresses and shear stresses in the wall and at the wall base due to lateral earthquake forces shall be computed on the basis of slab action (Sections 5.2 and 5.3) using pressure distribution consistent with the provisions of Section 5.3.1.

6.2—Circular tanks

The vertical bending stresses and shear stresses in the wall and at the wall base due to lateral earthquake forces shall be computed on the basis of shell action using an acceptable pressure distribution.

Hydrodynamic membrane (hoop) forces in the cylindrical wall corresponding to any liquid level y above the tank base shall be determined by

$$N_y = \sqrt{(N_{iy} + N_{wy})^2 + N_{cy}^2 + N_{hy}^2} \tag{6-1}$$

and hoop stress

$$\sigma_y = \frac{N_y}{12t_w} \tag{6-2}$$

$$\left[\sigma_y = \frac{N_y}{t_w} \text{ in the SI system}\right]$$

where t_w = wall thickness at the level being investigated (liquid level y).

R6.2—Circular tanks

For free-base circular tanks (Type 2.3), the terms in Eq. (6-1) are defined as

$$N_{iy} = p_{iy}\, r = \frac{2P_{iy}}{\pi} \text{ for (at } \theta = 0)$$

$$N_{cy} = p_{cy}\, r = \frac{16P_{cy}}{9\pi} \text{ for (at } \theta = 0)$$

$$N_{wy} = p_{wy}\, r = \frac{P_{wy}}{\pi}$$

$$N_{hy} = \ddot{u}_v Q_{hy}$$

where

$$Q_{hy} = q_{hy}\, r$$

For fixed- or hinged-base circular tanks (Types 2.1 and 2.2), the terms in Eq. (6-1) should be modified to account for the effects of base restraint. Similarly, the terms in Eq. (6-1) should be modified to account for the restraint of rigid wall-to-roof joints.

Notes

CHAPTER 7—FREEBOARD

STANDARD

7.1—Wave oscillation

Provisions shall be made to accommodate the maximum wave oscillation d_{max} generated by earthquake acceleration.

The maximum vertical displacement d_{max} to be accommodated shall be calculated from the following expressions:

Rectangular tanks

$$d_{max} = \frac{L}{2}C_cI \quad (7\text{-}1)$$

Circular tanks

$$d_{max} = \frac{D}{2}C_cI \quad (7\text{-}2)$$

where C_c is the seismic response coefficient as computed in Section 9.4.

COMMENTARY

R7.1—Wave oscillation

The horizontal earthquake acceleration causes the contained fluid to slosh with vertical displacement of the fluid surface.

The amount of freeboard required in design to accommodate this sloshing will vary. Where overtopping is tolerable, no freeboard provision is necessary. Where loss of liquid should be prevented (for example, tanks for the storage of toxic liquids) or where overtopping may result in scouring of the foundation materials or cause damage to pipes, roof, or both, then one or more of the following measures should be undertaken:

- Provide a freeboard allowance;
- Design the roof structure to resist the resulting uplift pressures; and/or
- Provide an overflow spillway.

Where site-specific response spectra are used, the maximum vertical displacement d_{max} may be calculated from the following expressions:

Rectangular tanks

$$d_{max} = \left(\frac{L}{2}\right)(C_cI) = \left(\frac{L}{2}\right)I(\eta_c)\frac{(0.667S_D)}{g}\left(\frac{2\pi}{T_c}\right)^2$$

Circular tanks

$$d_{max} = \left(\frac{D}{2}\right)(C_cI) = \left(\frac{D}{2}\right)I(\eta_c)\frac{(0.667S_D)}{g}\left(\frac{2\pi}{T_c}\right)^2$$

where C_c is defined in Section 9.5; and S_{cM} (as in the equation for C_c), η_c, and S_D are as defined in Section R4.2.2.

Notes

CHAPTER 8—EARTHQUAKE-INDUCED EARTH PRESSURES

STANDARD

8.1—General

Dynamic earth pressures shall be taken into account when computing the base shear of a partially or fully buried liquid-containing structure and when designing the walls.

The effects of groundwater table, if present, shall be included in the calculation of these pressures.

The coefficient of lateral earth pressure at rest K_o shall be used in estimating the earth pressures unless it is demonstrated by calculation that the structure deflects sufficiently to lower the coefficient to some value between K_o and the active coefficient of lateral earth pressure K_a.

In a pseudostatic analysis, the resultant of the seismic component of the earth pressure shall be assumed to act at a point 0.6 of the earth height above the base, and when part or all of the structure is below the water table, the resultant of the incremental increase in groundwater pressure shall be assumed to act at a point 1/3 of the water depth above the base.

8.2—Limitations

In a buried tank, the dynamic backfill forces shall not be relied upon to reduce the dynamic effects of the stored liquid or vice versa.

8.3—Alternative methods

The provisions of this chapter shall be permitted to be superseded by recommendations of the project geotechnical engineer that are approved by the building official having jurisdiction.

COMMENTARY

R8.1—General

The lateral forces due to the dynamic earth and groundwater pressures are combined algebraically with the impulsive forces on the tank as in Eq. (4-5).

Notes

CHAPTER 9—DYNAMIC MODEL

STANDARD

9.1—General

The dynamic characteristics of ground-supported liquid-containing structures subjected to earthquake acceleration shall be computed in accordance with Sections 9.2, 9.3 and 9.5.

The dynamic characteristics of pedestal-mounted liquid-containing structures shall be computed in accordance with Section 9.7.

COMMENTARY

R9.1—General

The lateral seismic pressures and forces determined in accordance with this Standard are based on vertical tank walls and vertical wall elements, and the pressures and forces may need to be modified for sloping surfaces.

The following commentary is adapted from Housner (1963):

The design procedures described in Chapter 4 recognize that the seismic analysis of liquid-containing structures subjected to a horizontal acceleration should include the inertia forces generated by the acceleration of the structure itself; and the hydrodynamic forces generated by the horizontal acceleration of the contained liquid.

According to Housner (1963), the pressures associated with these forces "can be separated into impulsive and convective parts. The impulsive pressures are not impulses in the usual sense but are associated with inertia forces produced by accelerations of the walls of the container and are directly proportional to these accelerations. The convective pressures are those produced by the oscillations of the fluid and are therefore the consequences of the impulsive pressures."

Figure R9.1 (on p. 43) shows an equivalent dynamic model for calculating the resultant seismic forces acting on a ground-based fluid container with rigid walls. This model has been accepted by the profession since the early 1960s. In this model, W_i represents the resultant effect of the impulsive seismic pressures on the tank walls. W_c represents the resultant of the sloshing (convective) fluid pressures.

In the model, W_i is rigidly fastened to the tank walls at a height h_i above the tank bottom, which corresponds to the location of the resultant impulsive force P_i. W_i moves with the tank walls as they respond to the ground shaking (the fluid is assumed to be incompressible and the fluid displacements small). The impulsive pressures are generated by the seismic accelerations of the tank walls so that the force P_i is evenly divided into a pressure force on the wall accelerating into the fluid, and a suction force on the wall accelerating away from the fluid. During an earthquake, the force P_i changes direction several times per second, corresponding to the change in direction of the base acceleration; the overturning moment generated by P_i is thus frequently ineffective in tending to overturn the tank.

W_c is the equivalent weight of the oscillating fluid that produces the convective pressures on the tank walls with resultant force P_c, which acts at a height of h_c above the tank bottom. In the model, W_c is fastened to the tank walls by springs

STANDARD

COMMENTARY

that produce a period of vibration corresponding to the period of fluid sloshing. The sloshing pressures on the tank walls result from the fluid motion associated with the wave oscillation. The period of oscillation of the sloshing depends on the ratio of fluid depth to tank diameter, and is usually several seconds. The overturning moment exerted by P_c (Fig. R9.1 [on p. 43]) acts for a sufficient time to tend to uplift the tank wall if there is insufficient restraining weight. The forces P_i and P_c act independently and simultaneously on the tank. The force P_i (and its associated pressures) primarily act to stress the tank wall, whereas P_c acts primarily to uplift the tank wall. The vertical vibrations of the ground are also transmitted to the fluid, thus producing pressures that act on the tank walls. They act to increase or decrease the hoop stresses.

The pressures and forces on a cylindrical tank are similar to, but not the same as, those acting on a rectangular tank.

The rapid fluctuations of the force P_i mean that the bending moments and stresses in the wall of a rectangular tank also vary rapidly (the effect is not like a constant force acting on the wall). The duration of the fluctuations is 10 to 15 seconds for earthquakes of magnitude 6.5 to 7.5.

The force P_c fluctuates sinusoidally with a period of vibration that depends on the dimensions of the tank, and can be several seconds or longer. The duration of sloshing can be 20 to 40 seconds for earthquakes of magnitude 6.5 to 7.5. Note that the damping of the sloshing water is small: approximately 0.5 to 1% of critical damping. The sloshing increases and decreases the fluid pressure on the wall. Normally, this is smaller than the impulsive effect, but if there is not enough dead load, the tank will tend to uplift.

9.2—Rectangular tanks (Type 1)

9.2.1—Equivalent weights of accelerating liquid (Fig. 9.2.1 [on p. 44])

$$\frac{W_i}{W_L} = \frac{\tanh\left[0.866\left(\frac{L}{H_L}\right)\right]}{0.866\left(\frac{L}{H_L}\right)} \qquad (9\text{-}1)$$

$$\frac{W_c}{W_L} = 0.264\left(\frac{L}{H_L}\right)\tanh\left[3.16\left(\frac{H_L}{L}\right)\right] \qquad (9\text{-}2)$$

9.2.2—Height to centers of gravity EBP (Fig. 9.2.2 [on p. 45])

For tanks with $\frac{L}{H_L} < 1.333$

R9.2—Rectangular tanks (Type 1)

All equations in Section 9.2 except Eq. (9-9), (9-10), and (9-11) were originally developed by Housner (1963), and subsequently used by other authors (Housner 1956; NZS 1986; Haroun 1984; ASCE 1981; Veletsos and Shivakumar 1997; ANSI/AWWA 1995a,b; Haroun and Ellaithy 1985).

Equations (9-9), (9-10), and (9-11) were adapted from NZS (1986).

STANDARD

$$\frac{h_i}{H_L} = 0.5 - 0.09375\left(\frac{L}{H_L}\right) \qquad (9\text{-}3)$$

For tanks with $\frac{L}{H_L} \geq 1.333$

$$\frac{h_i}{H_L} = 0.375 \qquad (9\text{-}4)$$

For all tanks

$$\frac{h_c}{H_L} = 1 - \frac{\cosh\left[3.16\left(\frac{H_L}{L}\right)\right] - 1}{3.16\left(\frac{H_L}{L}\right)\sinh\left[3.16\left(\frac{H_L}{L}\right)\right]} \qquad (9\text{-}5)$$

9.2.3—Heights to center of gravity IBP (Fig. 9.2.3 [on p. 46])

For tanks with $\frac{L}{H_L} < 0.75$

$$\frac{h_i'}{H_L} = 0.45 \qquad (9\text{-}6)$$

For tanks with $\frac{L}{H_L} \geq 0.75$

$$\frac{h_i'}{H_L} = \frac{0.866\left(\frac{L}{H_L}\right)}{2\tanh\left[0.866\left(\frac{L}{H_L}\right)\right]} - \frac{1}{8} \qquad (9\text{-}7)$$

For all tanks

$$\frac{h_c'}{H_L} = 1 - \frac{\cosh\left[3.16\left(\frac{H_L}{L}\right)\right] - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left[3.16\left(\frac{H_L}{L}\right)\right]} \qquad (9\text{-}8)$$

9.2.4—Dynamic properties

The structural stiffness k shall be computed on the basis of correct boundary conditions.

$$\omega_i = \sqrt{\frac{k}{m}} \qquad (9\text{-}9)$$

$$m = m_w + m_i \qquad (9\text{-}10)$$

COMMENTARY

R9.2.4—Dynamic properties

The following equations are provided as examples for the special case of a wall of uniform thickness. (Note that mass is equal to weight divided by the acceleration due to gravity.)

$$m_w = H_w \frac{t_w}{12}\left(\frac{\gamma_c}{g}\right)$$

STANDARD

$$T_i = \frac{2\pi}{\omega_i} = 2\pi\sqrt{\frac{m}{k}} \qquad (9\text{-}11)$$

$$\omega_c = \frac{\lambda}{\sqrt{L}} \qquad (9\text{-}12)$$

where

$$\lambda = \sqrt{3.16g\tanh\left[3.16\left(\frac{H_L}{L}\right)\right]} \qquad (9\text{-}13)$$

$$T_c = \frac{2\pi}{\omega_c} = \left(\frac{2\pi}{\lambda}\right)\sqrt{L} \qquad (9\text{-}14)$$

$$\left[\left(\frac{2\pi}{\lambda}\right) \text{ from Fig. 9.2.4}\right]$$

9.3—Circular tanks (Type 2)

9.3.1—Equivalent weights of accelerating liquid (Fig. 9.3.1 [on p. 48])

$$\frac{W_i}{W_L} = \frac{\tanh\left[0.866\left(\frac{D}{H_L}\right)\right]}{0.866\left(\frac{D}{H_L}\right)} \qquad (9\text{-}15)$$

$$\frac{W_c}{W_L} = 0.230\left(\frac{D}{H_L}\right)\tanh\left[3.68\left(\frac{H_L}{D}\right)\right] \qquad (9\text{-}16)$$

COMMENTARY

$$\left[m_w = H_w\frac{t_w}{10^3}\left(\frac{\gamma_c}{g}\right) \text{ in the SI system}\right]$$

$$m_i = \left(\frac{W_i}{W_L}\right)\left(\frac{L}{2}\right)H_L\left(\frac{\gamma_L}{g}\right)$$

$$h = \frac{(h_w m_w + h_i m_i)}{(m_w + m_i)}$$

where $h_w = 0.5H_w$, and h_i is obtained from Eq. (9-3) and (9-4), and Fig. 9.2.2 (on p. 44).

For walls of nonuniform thickness, special analysis is required to determine m_w, m_i, and h.

For fixed-base, free-top cantilever walls, such as in open-top tanks, flexural stiffness for a unit width of wall k may be approximated using the following equation

$$k = \frac{E_c}{48}\left(\frac{t_w}{h}\right)^3$$

$$\left[k = \frac{E_c}{4 \times 10^6}\left(\frac{t_w}{h}\right)^3 \text{ in the SI system}\right]$$

Flexural stiffness formulas may be developed for other wall support conditions. Such spring constants will generally fall within the low period range (less than about 0.3 seconds) for tanks of normal proportions.

As an alternative to computing the natural period of vibration, particularly for end conditions other than cantilever, it is reasonable to assume the wall rigid. In such a case, Eq. (9-32) may be conservatively used to calculate the impulsive forces regardless of the actual boundary conditions of the structure or structural components being analyzed.

R9.3—Circular tanks (Type 2)

All equations in Section 9.3, except Eq. (9-23) through (9-28), were originally developed by Housner (1963), and subsequently used by other authors (Housner 1956; NZS 1986; Haroun 1984; ASCE 1981; Veletsos and Shivakumar 1997; ANSI/AWWA 1995a,b; Haroun and Ellaithy 1985).

Equations (9-23) through (9-28) were adapted from NZS (1986).

STANDARD

9.3.2—Heights to centers of gravity (EBP [Fig. 9.3.2; on p. 49])

For tanks with $\frac{D}{H_L} < 1.333$

$$\frac{h_i}{H_L} = 0.5 - 0.09375\left(\frac{D}{H_L}\right) \tag{9-17}$$

For tanks with $\frac{L}{H_L} \geq 1.333$

$$\frac{h_i}{H_L} = 0.375 \tag{9-18}$$

For all tanks

$$\frac{h_c}{H_L} = 1 - \frac{\cosh\left[3.68\left(\frac{H_L}{D}\right)\right] - 1}{3.68\left(\frac{H_L}{D}\right)\sinh\left[3.68\left(\frac{H_L}{D}\right)\right]} \tag{9-19}$$

9.3.3—Heights to center of gravity (IBP [Fig. 9.3.3; on p. 50])

For tanks with $\frac{D}{H_L} < 0.75$

$$\frac{h_i'}{H_L} = 0.45 \tag{9-20}$$

For tanks with $\frac{D}{H_L} \geq 0.75$

$$\frac{h_i'}{H_L} = \frac{0.866\left(\frac{D}{H_L}\right)}{2\tanh\left[0.866\left(\frac{D}{H_L}\right)\right]} - \frac{1}{8} \tag{9-21}$$

$$\frac{h_c'}{H_L} = 1 - \frac{\cosh\left[3.68\left(\frac{H_L}{D}\right)\right] - 2.01}{3.68\left(\frac{H_L}{D}\right)\sinh\left[3.68\left(\frac{H_L}{D}\right)\right]} \tag{9-22}$$

9.3.4—Dynamic properties

***T_i*: For tank Types 2.1 and 2.2:**

$$\omega_i = C_l \frac{12}{H_L}\sqrt{E_c \frac{g}{\gamma_c}} \tag{9-23}$$

COMMENTARY

R9.3.4—Dynamic properties

Equations (9-23) and (9-24) are adapted from ASCE (1981) and Veletsos and Shivakumar (1997).

Equations (9-26) and (9-27) are adapted from ANSI/AWWA (1995a,b).

STANDARD

$$[\omega_i = C_l \frac{1}{H_L}\sqrt{10^3 E_c \frac{g}{\gamma_c}} \text{ in the SI system}]$$

$$C_l = C_w 10\sqrt{\frac{t_w}{12r}} \qquad (9\text{-}24)$$

[C_w from Fig. 9.3.4(a); on p. 51]

$$[C_l = C_w\sqrt{\frac{t_w}{10r}} \text{ in the SI system}]$$

$$T_i = \frac{2\pi}{\omega_i} \qquad (9\text{-}25)$$

For tank Type 2.3

$$T_i = \sqrt{\frac{8\pi(W_w + W_r + W_i)}{gDk_a}} \qquad (9\text{-}26)$$

but shall not exceed 1.25 seconds.

$$k_a = 144\left[\left(\frac{A_s E_s \cos^2\alpha}{L_c S_c}\right) + \left(\frac{2G_p w_p L_p}{t_p S_p}\right)\right] \qquad (9\text{-}27)$$

$$k_a = 10^3\left[\left(\frac{A_s E_s \cos^2\alpha}{L_c S_c}\right) + \left(\frac{2G_p w_p L_p}{t_p S_p}\right)\right] \text{ [in the SI system]}$$

T_c:

$$\omega_c = \frac{\lambda}{\sqrt{D}} \qquad (9\text{-}28)$$

where

$$\lambda = \sqrt{3.68 g \tanh\left[3.68\left(\frac{H_L}{D}\right)\right]} \qquad (9\text{-}29)$$

$$T_c = \frac{2\pi}{\omega_c} = \left(\frac{2\pi}{\lambda}\right)\sqrt{D} \qquad (9\text{-}30)$$

[$\left(\frac{2\pi}{\lambda}\right)$ from Fig. 9.3.4(b); on p. 52]

T_v: For circular tanks

$$T_v = 2\pi\sqrt{\frac{Y_L D H_L^2}{24 g t_w E_c}} \qquad (9\text{-}31)$$

COMMENTARY

Equations (9-13), (9-14), (9-29), and (9-30) are adapted from Housner (1963).

STANDARD

$$[\, T_v = 2\pi\sqrt{\frac{Y_L D H_L^2}{2 g t_w E_c}} \text{ in the SI system}]$$

9.4—Seismic response coefficients C_i, C_c, and C_t

9.4.1 C_i shall be determined as follows:

For $T_I \le T_S$

$$C_i = S_{DS} \tag{9-32}$$

For $T_i > T_S$

$$C_i = \frac{S_{D1}}{T_i} \le S_{DS} \tag{9-33}$$

where

$$T_S = \frac{S_{D1}}{S_{DS}} \tag{9-34}$$

S_{DS} = the design spectral response acceleration at short periods

$$S_{DS} = \frac{2}{3} S_S F_a \tag{9-35}$$

S_{D1} = the design spectral response acceleration at a 1-second period

COMMENTARY

R9.4—Seismic response coefficients C_i, C_c, and C_t

In practice, Designations C_i, C_c, and C_t define the profile of the design response spectrum at periods T_i, T_c, and T_v, respectively. A plot of the seismic response coefficient C_i is shown in the design response spectrum in Fig. R9.4.1, which is adapted from IBC (2003).

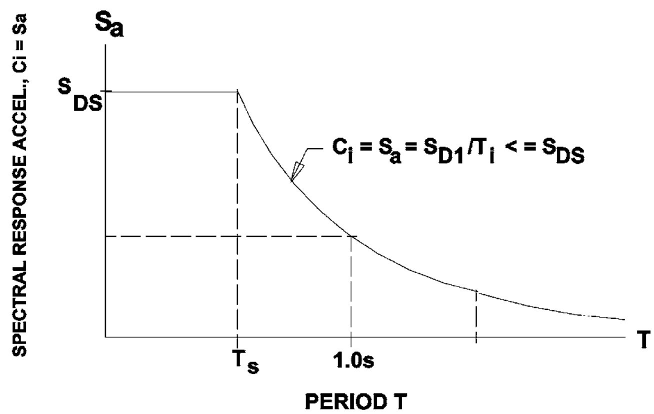


Fig. R9.4.1—Design response spectrum.

R9.4.1 The mapped spectral response accelerations S_s and S_1 for any location can also be obtained from the latest database of the U.S. Geological Survey (USGS), at http://eqhazmaps.usgs.gov, using the specific zip code or latitude and longitude that identify the location.

In regions other than those shown in the maps in publications by IBC (2003), Building Seismic Safety Council (1997, 2000), and ASCE (2005), S_S and S_1 may be replaced by the maximum considered earthquake spectral response accelerations from 5% damped response spectra representing earthquakes with a 2% probability of exceedance in a 50-year period, equivalent to a recurrence interval of approximately 2500 years.

STANDARD

$$S_{D1} = \frac{2}{3}S_1F_v \qquad (9\text{-}36)$$

S_S and S_1 are the mapped spectral response accelerations at short periods (S_s) and 1 second (S_1), respectively, and shall be obtained from the seismic ground motion maps in Fig. 22-1 through 22-14 of ASCE 7-05, Chapter 22; and F_a and F_v are the site coefficients and shall be obtained from Table 11.4-1 and 11.4-2, respectively, of ASCE 7-05, in conjunction with Table 20.3-1, "Site Classification," of ASCE 7-05.

9.4.2 C_c shall be determined as follows:

For $T_c \leq 1.6/T_S$ seconds

$$C_c = \frac{1.5S_{D1}}{T_c} \leq 1.5S_{DS} \qquad (9\text{-}37)$$

For $T_c > 1.6/T_S$ seconds

$$C_c = 6\frac{0.4S_{DS}}{T_c^2} = \frac{2.4S_{DS}}{T_c^2} \qquad (9\text{-}38)$$

9.4.3 C_t shall be determined as follows:

For circular tanks

For $T_v \leq T_S$

$$C_t = S_{DS} \qquad (9\text{-}39)$$

For $T_v > T_S$

$$C_t = \frac{S_{D1}}{T_v} \qquad (9\text{-}40)$$

For rectangular tanks, $C_t = 0.4S_{DS}$.

9.5—Site-specific seismic response coefficients C_i, C_c, and C_t

When site-specific procedures are used, the maximum considered earthquake spectral response accelerations S_{aM} and S_{cM} shall be obtained from the site-specific acceleration spectrum as follows:

For periods less than or equal to T_S, S_{aM} shall be taken as the spectral acceleration obtained from the site-specific spectra at a period of 2 seconds, except that it shall not be taken less than 90% of the peak spectral acceleration at any period larger than 0.2 seconds. For

COMMENTARY

R9.4.2 Factor 1.5 represents the approximate ratio of the spectral amplifications based on 0.5% damping to those based on 5% damping. $0.4S_{DS}$ in Eq. (9-38) is an approximation of the effective peak ground acceleration S_0 (at $T = 0$) reduced by a factor of 2/3.

R9.4.3 The period of vibration of vertical liquid motion T_v for a circular tank (upright cylinder) is derived from the axisymmetric pulsating ("breathing") of the cylindrical wall due to the hydrodynamic pressures resulting from the vertical, piston-like "pounding" of the stored liquid by the vertically accelerating ground.

This mode of vibration is relevant only to circular tanks, and does not apply to rectangular tanks. While the derivation of T_v for circular tanks has been the subject of several technical papers, the committee is not aware of any work devoted to the derivation of this parameter for rectangular tanks. Therefore, for rectangular tanks, C_t is taken independent of the period of vibration.

R9.5—Site-specific seismic response coefficients C_i, C_c, and C_t

For damping ratios other than 5% of critical, refer to Section R4.2.

If the site-specific response spectrum does not extend into, or is not well defined in the T_c range, coefficient C_c may be calculated using the equation

$$C_c = 6\frac{\frac{2}{3}S_0}{T_c^2} = \frac{4S_0}{T_c^2}$$

STANDARD

periods greater than T_S, S_{aM} shall be taken as the spectral response acceleration corresponding to T_i or T_v, as applicable. When a 5% damped, site-specific vertical response spectrum is available, S_{aM} shall be determined from that spectrum when used to determine C_t; and

S_{cM} shall be taken as 150% of the spectral response acceleration corresponding to T_c, except that when a 0.5% damped, site-specific horizontal response spectrum is available, S_{cM} shall be equal to the spectral response acceleration from that spectrum corresponding to period T_c.

The seismic response coefficients C_i, C_c, and C_t shall be determined from Eq. (9-41), (9-42), and (9-43), respectively.

For all periods, T_i

$$C_i = \frac{2}{3}S_{aM} \tag{9-41}$$

For all periods, T_c

$$C_c = \frac{2}{3}S_{cM} \tag{9-42}$$

For all periods, T_v

$$C_t = \frac{2}{3}S_{aM} \tag{9-43}$$

The values of C_i, C_c, and C_t used for design shall not be less than 80% of the corresponding values as determined in accordance with Section 9.4.

9.6—Effective mass coefficient ε

9.6.1—Rectangular tanks

$$\varepsilon = \left[0.0151\left(\frac{L}{H_L}\right)^2 - 0.1908\left(\frac{L}{H_L}\right) + 1.021\right] \leq 1.0 \tag{9-44}$$

9.6.2—Circular tanks

$$\varepsilon = \left[0.0151\left(\frac{D}{H_L}\right)^2 - 0.1908\left(\frac{D}{H_L}\right) + 1.021\right] \leq 1.0 \tag{9-45}$$

COMMENTARY

where S_0 is the effective site-specific peak ground acceleration (at $T = 0$) expressed as a fraction of the acceleration due to gravity g.

The use of site-specific response spectra represents one specific case of an “accepted alternative method of analysis” permitted in Section 21.2.1.7 of ACI 350-06. Therefore, the 80% lower limit imposed in 9.5 should be considered the same as the limit imposed in Section 21.2.1.7 of ACI 350-06.

R9.6—Effective mass coefficient ε

The coefficient ε represents the ratio of the equivalent (or generalized) dynamic mass of the tank shell to its actual total mass. Equation (9-44) and (9-45) are adapted from ASCE (1981).

For additional information related to the effective mass coefficient ε, consult Veletsos and Shivakumar (1997).

STANDARD

9.7—Pedestal-mounted tanks

The equivalent weights, W_i and W_c , and heights to the centers of gravity, h_i, h_c, h_i', and h_c' of a mounted tank, shall be computed using the corresponding Eq. (9-2) and (9-3) for rectangular and circular tanks, respectively.

The dynamic properties, including periods of vibration and lateral coefficients, shall be permitted to be determined on the basis of generally acceptable methods of dynamic analysis.

COMMENTARY

R9.7—Pedestal-mounted tanks

Housner (1963), Haroun and Ellaithy (1985), and ACI Committee 371 (1998) provide additional guidelines on the dynamic analysis of pedestal-mounted tanks.

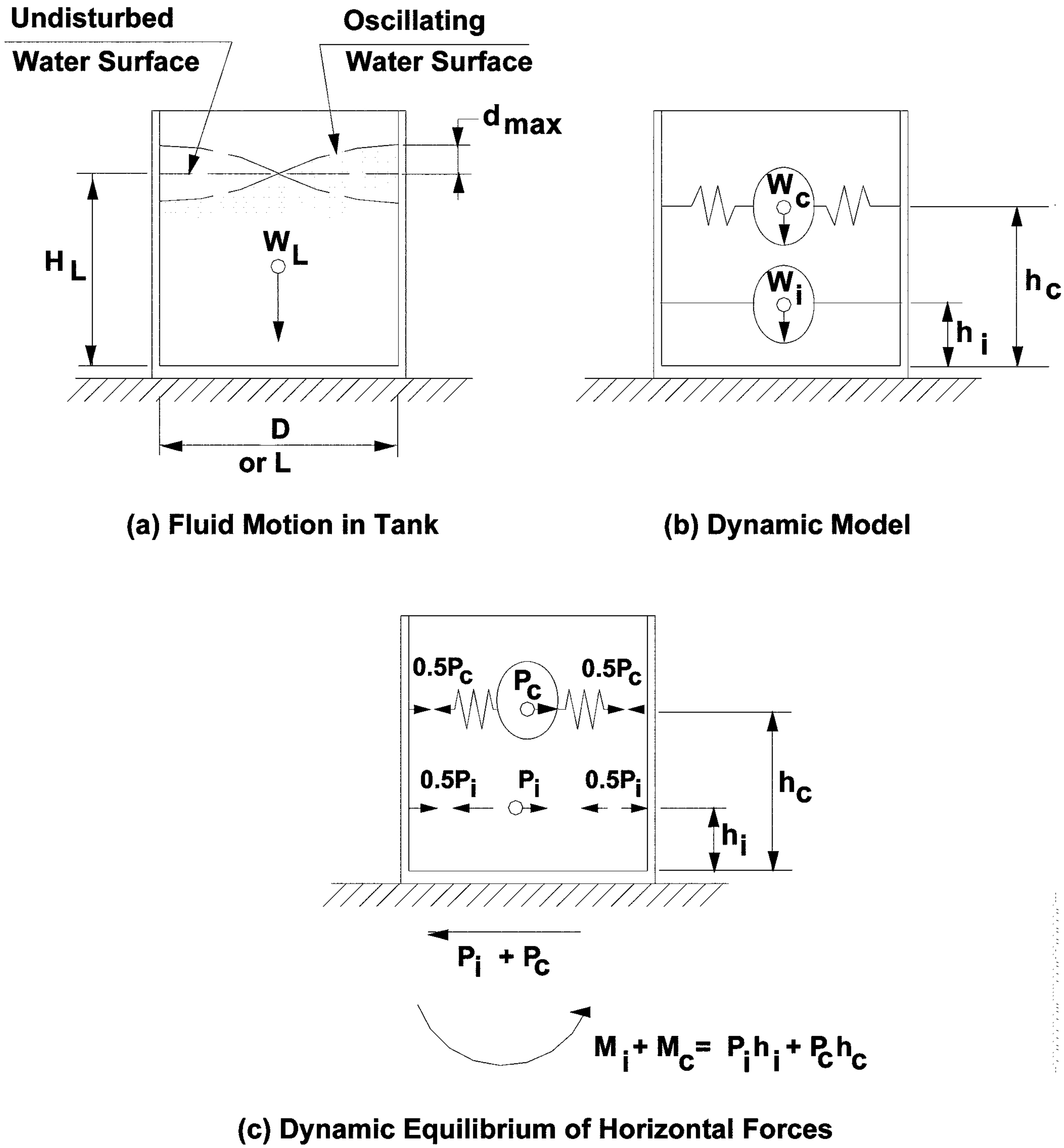


Fig. R9.1—Dynamic model of liquid-containing tank rigidly supported on the ground (adapted from Housner [1963] and ASCE [1984]).

STANDARD COMMENTARY

IMPULSIVE AND CONVECTIVE MASS FACTORS vs. L/H_L RATIO

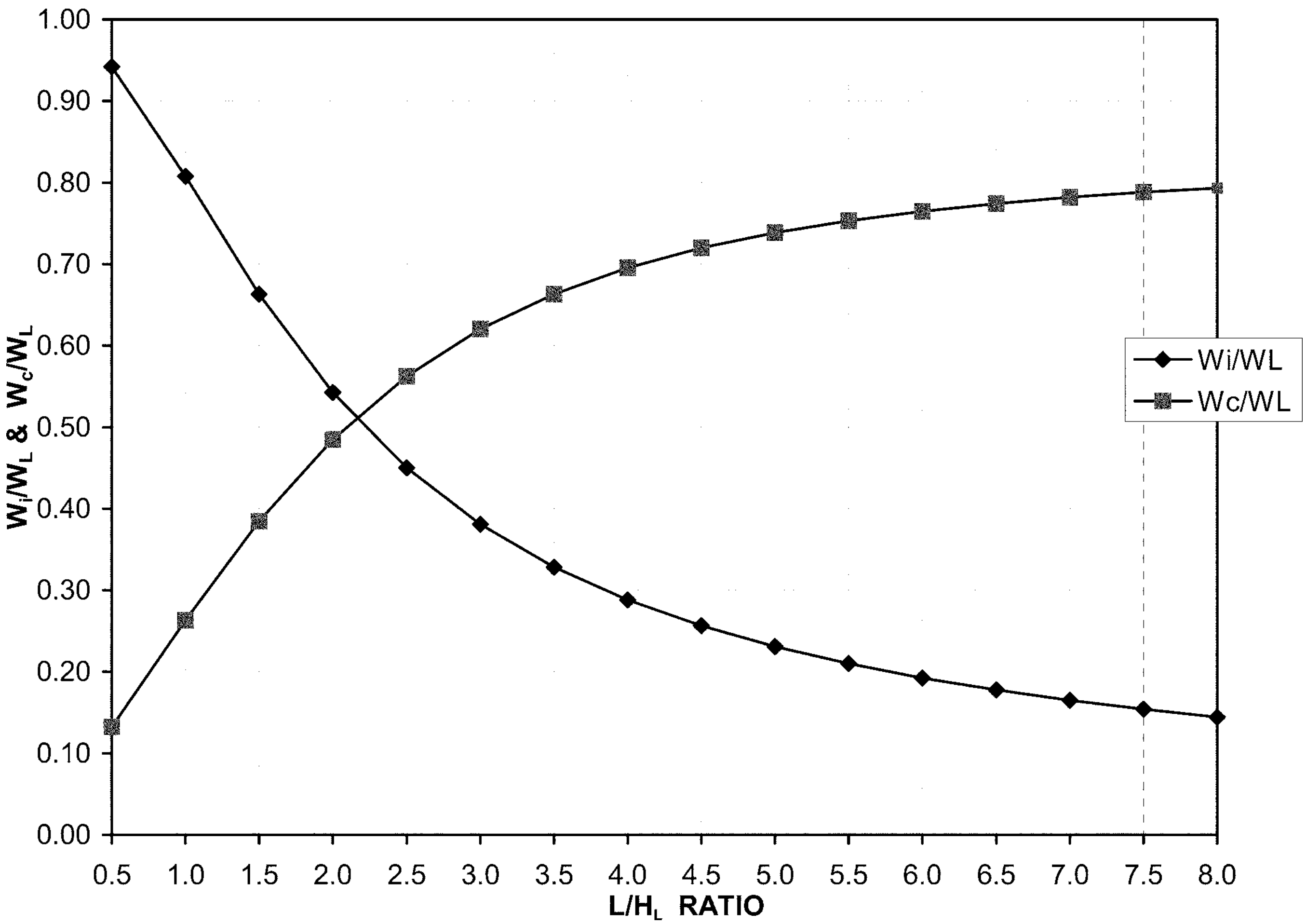


Fig. 9.2.1—Factors W_i/W_L *and* W_c/W_L *versus ratio* L/H_L *for rectangular tanks.*

$$\frac{W_i}{W_L} = \frac{\tanh\left[0.866\left(\frac{L}{H_L}\right)\right]}{0.866\left(\frac{L}{H_L}\right)} \tag{9-1}$$

$$\frac{W_c}{W_L} = 0.264\left(\frac{L}{H_L}\right)\tanh\left[3.16\left(\frac{H_L}{L}\right)\right] \tag{9-2}$$

STANDARD **COMMENTARY**

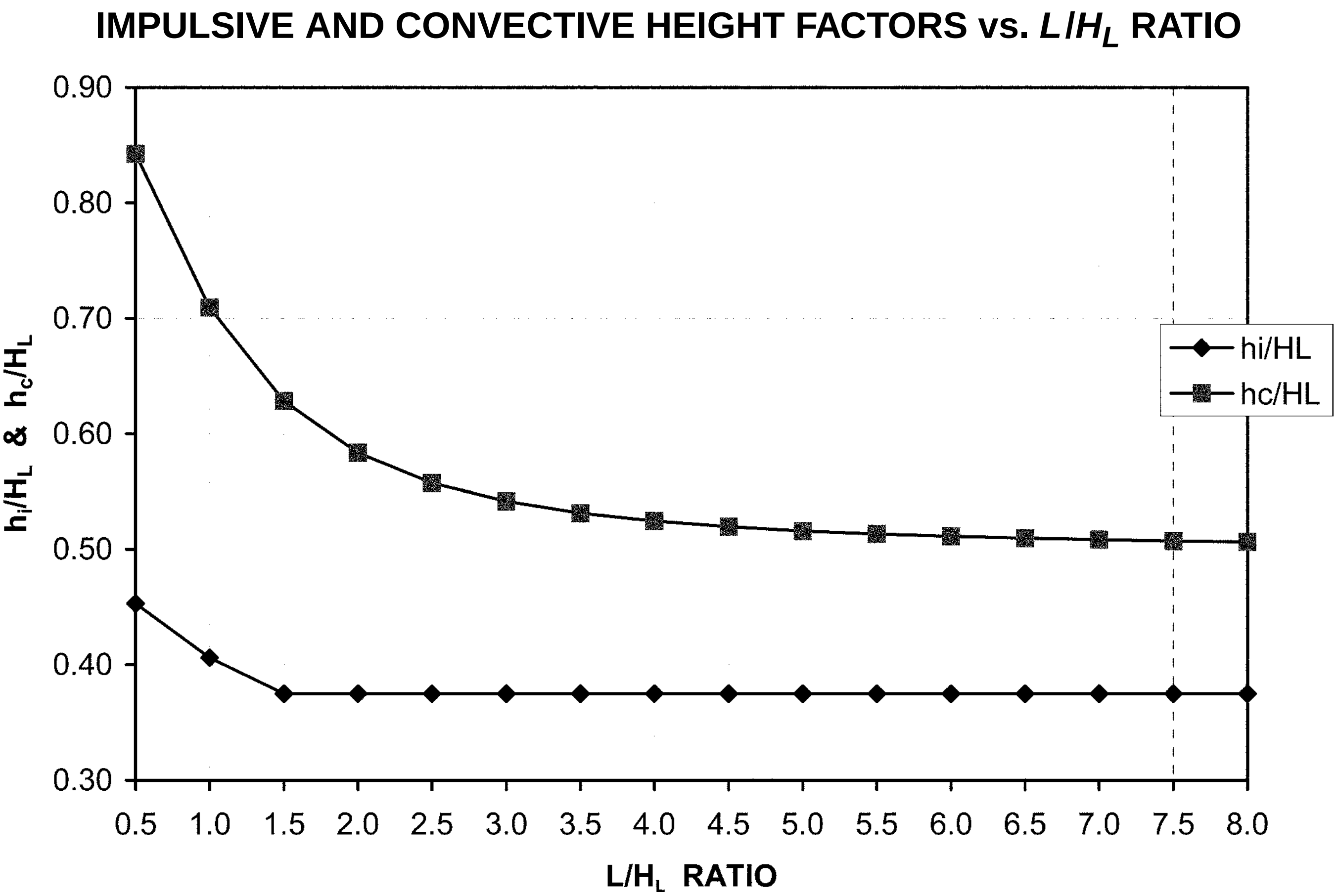


Fig. 9.2.2—Factors $\mathbf{h_i/H_L}$ *and* $\mathbf{h_c/H_L}$ *versus ratio* $\mathbf{L/H_L}$ *for rectangular tanks (EBP).*

h_i/H_L:

For tanks with $\frac{L}{H_L} < 1.333$ $\qquad \frac{h_i}{H_L} = 0.5 - 0.09375\left(\frac{L}{H_L}\right)$ (9-3)

For tanks with $\frac{L}{H_L} \geq 1.333$ $\qquad \frac{h_i}{H_L} = 0.375$ (9-4)

h_c/H_L:

For all tanks

$$\frac{h_c}{H_L} = 1 - \frac{\cosh\left[3.16\left(\frac{H_L}{L}\right)\right] - 1}{3.16\left(\frac{H_L}{L}\right) \times \sinh\left[3.16\left(\frac{H_L}{L}\right)\right]} \qquad (9\text{-}5)$$

STANDARD **COMMENTARY**

IMPULSIVE AND CONVECTIVE HEIGHT FACTORS vs. L/H_L RATIO

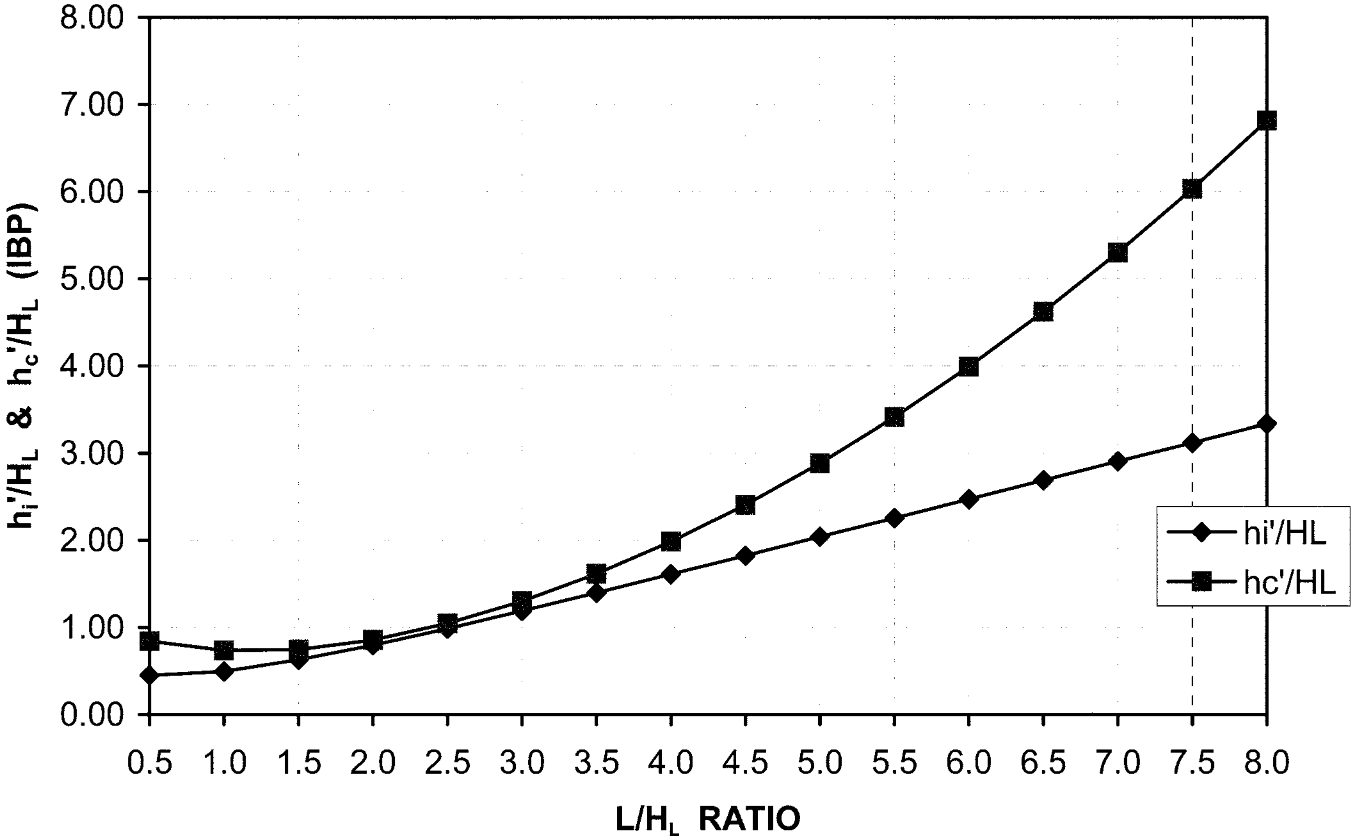


Fig. 9.2.3—Factors h_i'/H_L *and* h_c'/H_L *versus ratio* L/H_L *for rectangular tanks (IBP).*

h_i'/H_L:

For tanks with $\frac{L}{H_L} < 0.75$

$$\frac{h_i'}{H_L} = 0.45 \tag{9-6}$$

For tanks with $\frac{L}{H_L} \geq 0.75$

$$\frac{h_i'}{H_L} = \frac{0.866\left(\frac{L}{H_L}\right)}{2 \times \tanh\left[0.866\left(\frac{L}{H_L}\right)\right]} - \frac{1}{8} \tag{9-7}$$

h_c'/H_L:

For all tanks

$$\frac{h_c'}{H_L} = 1 - \frac{\cosh\left[3.16\left(\frac{H_L}{L}\right)\right] - 2.01}{3.16\left(\frac{H_L}{L}\right)\sinh\left[3.16\left(\frac{H_L}{L}\right)\right]} \tag{9-8}$$

STANDARD **COMMENTARY**

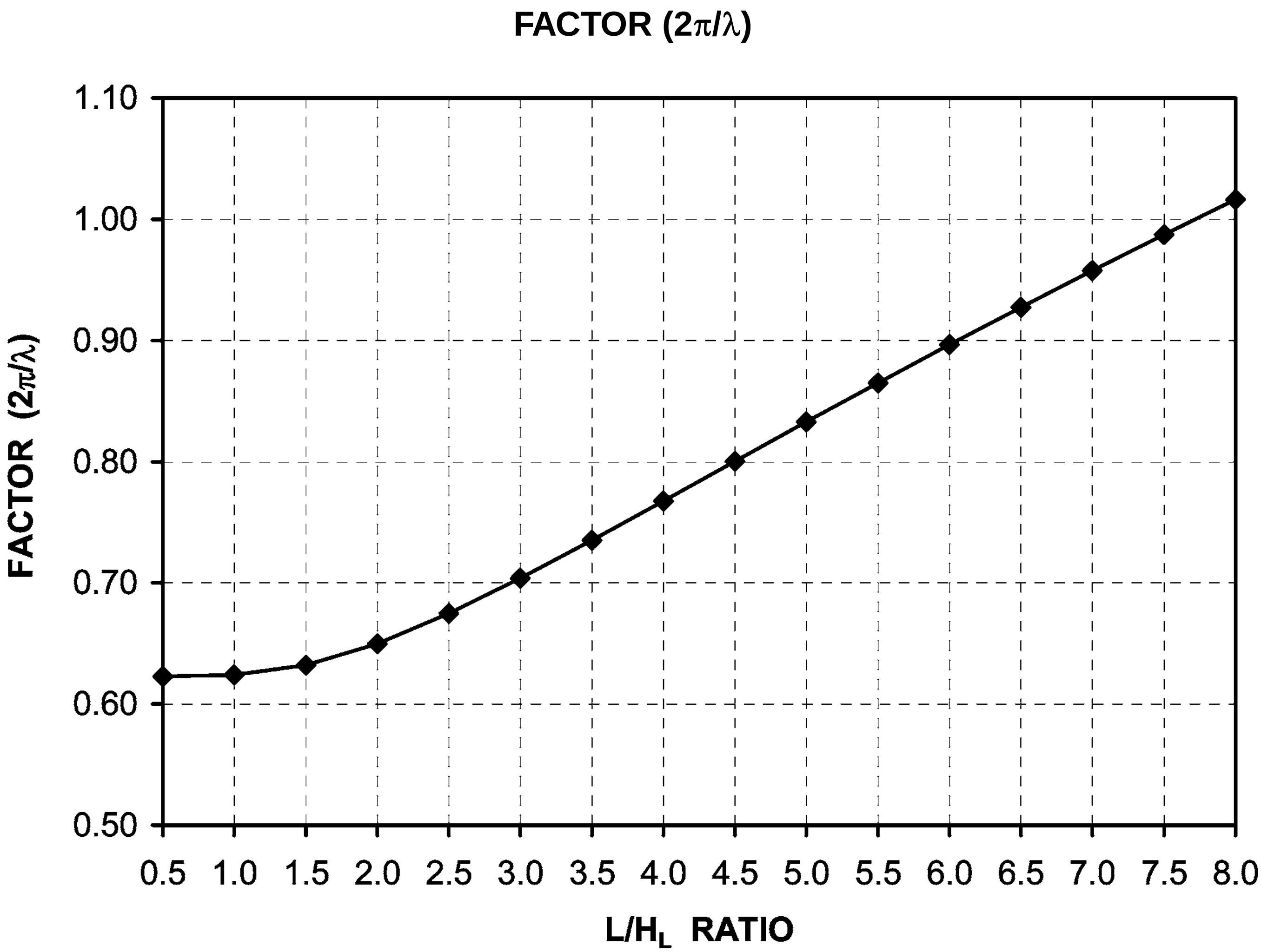


Fig. 9.2.4—Factor $\mathbf{2\pi/\lambda}$ *for rectangular tanks.*

$$\omega_c = \frac{\lambda}{\sqrt{L}} \tag{9-12}$$

$$\lambda = \sqrt{3.16 g \tanh\left[3.16\left(\frac{H_L}{L}\right)\right]} \tag{9-13}$$

$$T_c = \frac{2\pi}{\omega_c} = \left(\frac{2\pi}{\lambda}\right)\sqrt{L} \tag{9-14}$$

STANDARD | **COMMENTARY**

IMPULSIVE AND CONVECTIVE MASS FACTORS vs. D/H_L RATIO

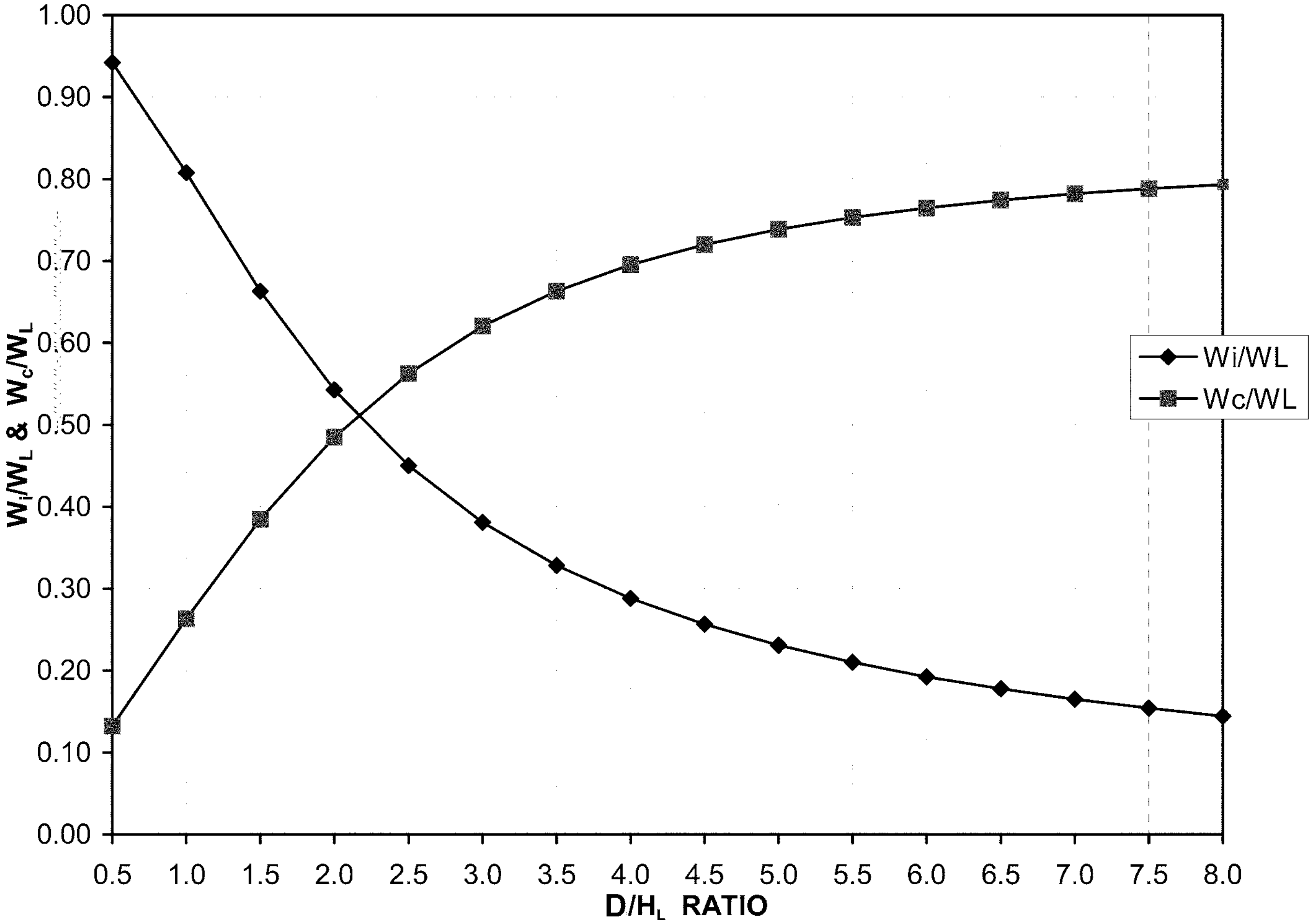


Fig. 9.3.1—Factors $\mathbf{W_i / W_L}$ *and* $\mathbf{W_c/W_L}$ *versus ratio* $\mathbf{D/H_L}$ *for circular tanks.*

$$\frac{W_i}{W_L} = \frac{\tanh\left[0.866\left(\frac{D}{H_L}\right)\right]}{0.866\left(\frac{D}{H_L}\right)} \tag{9-15}$$

$$\frac{W_c}{W_L} = 0.230\left(\frac{D}{H_L}\right)\tanh\left[3.68\left(\frac{H_L}{D}\right)\right] \tag{9-16}$$

STANDARD **COMMENTARY**

IMPULSIVE AND CONVECTIVE HEIGHT FACTORS vs. D/H_L RATIO

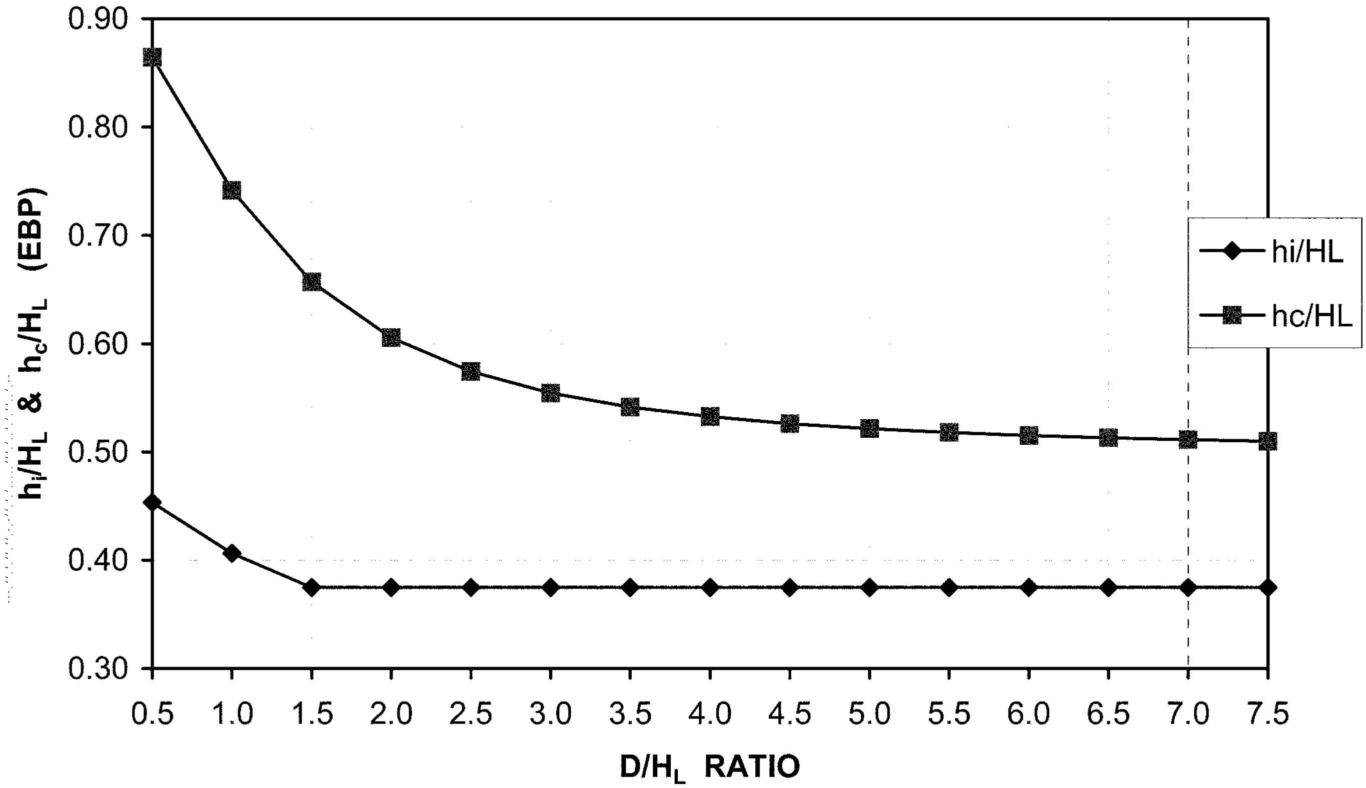


Fig. 9.3.2—Factors h_i/H_L *and* h_c/H_L *versus ratio* D/H_L *for circular tanks* (EBP).

h_i/H_L:

For tanks with $\frac{D}{H_L} < 1.333$

$$\frac{h_i}{H_L} = 0.5 - 0.09375\left(\frac{D}{H_L}\right) \tag{9-17}$$

For tanks with $\frac{D}{H_L} \geq 1.333$

$$\frac{h_i}{H_L} = 0.375 \tag{9-18}$$

h_c/H_L:

For all tanks

$$\frac{h_c}{H_L} = 1 - \frac{\cosh\left[3.68\left(\frac{H_L}{D}\right)\right] - 1}{3.68\left(\frac{H_L}{D}\right)\sinh\left[3.68\left(\frac{H_L}{D}\right)\right]} \tag{9-19}$$

STANDARD COMMENTARY

IMPULSIVE AND CONVECTIVE HEIGHT FACTORS vs. D/H_L RATIO

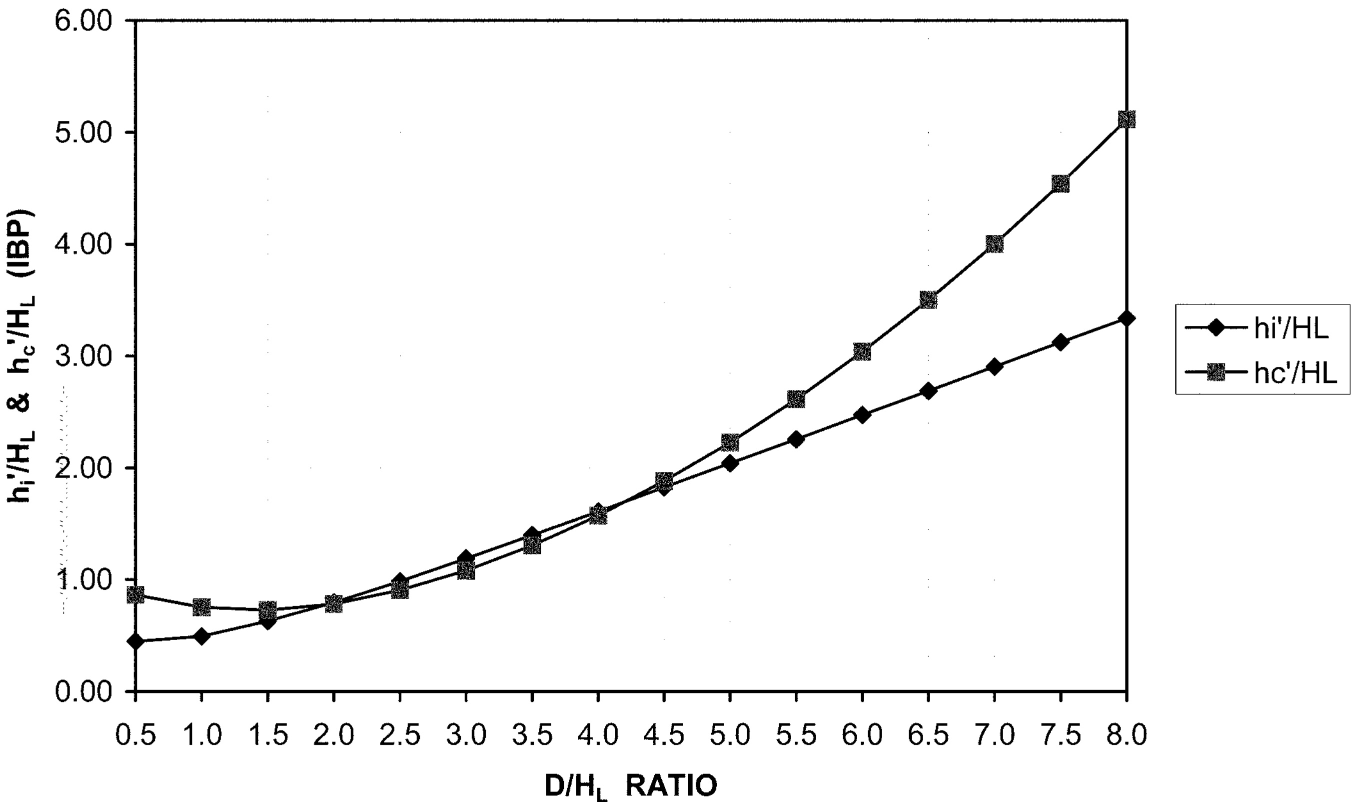


Fig. 9.3.3—Factors h_i'/H_L *and* h_c'/H_L *versus ratio* D/H_L *for circular tanks (IBP).*

h_i'/H_L:

For tanks with $\frac{D}{H_L} < 0.75$ $\qquad \frac{h_i'}{H_L} = 0.45$ (9-20)

For tanks with $\frac{D}{H_L} \geq 0.75$

$$\frac{h_i'}{H_L} = \frac{0.866\left(\frac{D}{H_L}\right)}{2\tanh\left[0.866\left(\frac{D}{H_L}\right)\right]} - \frac{1}{8} \tag{9-21}$$

h_c'/H_L:

For all tanks

$$\frac{h_c'}{H_L} = 1 - \frac{\cosh\left[3.68\left(\frac{H_L}{D}\right)\right] - 2.01}{3.68\left(\frac{H_L}{D}\right)\sinh\left[3.68\left(\frac{H_L}{D}\right)\right]} \tag{9-22}$$

STANDARD **COMMENTARY**

COEFFICIENT C_W

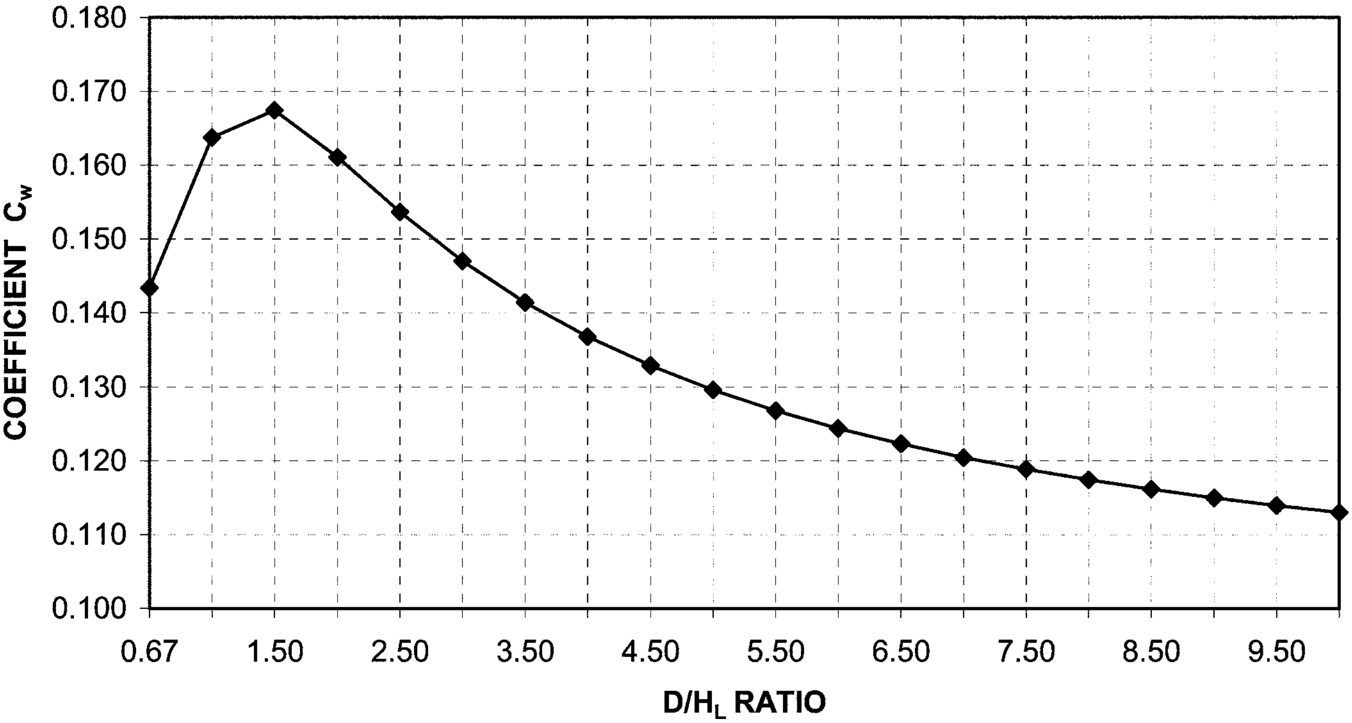


Fig. 9.3.4(a)—Coefficient $\mathbf{C_w}$ *for circular tanks.*

For $D/H_L > 0.667$

$$C_w = 9.375\times10^{-2} + 0.2039\left(\frac{H_L}{D}\right) - 0.1034\left(\frac{H_L}{D}\right)^2 - 0.1253\left(\frac{H_L}{D}\right)^3 + 0.1267\left(\frac{H_L}{D}\right)^4 - 3.186\times10^{-2}\left(\frac{H_L}{D}\right)^5$$

STANDARD

COMMENTARY

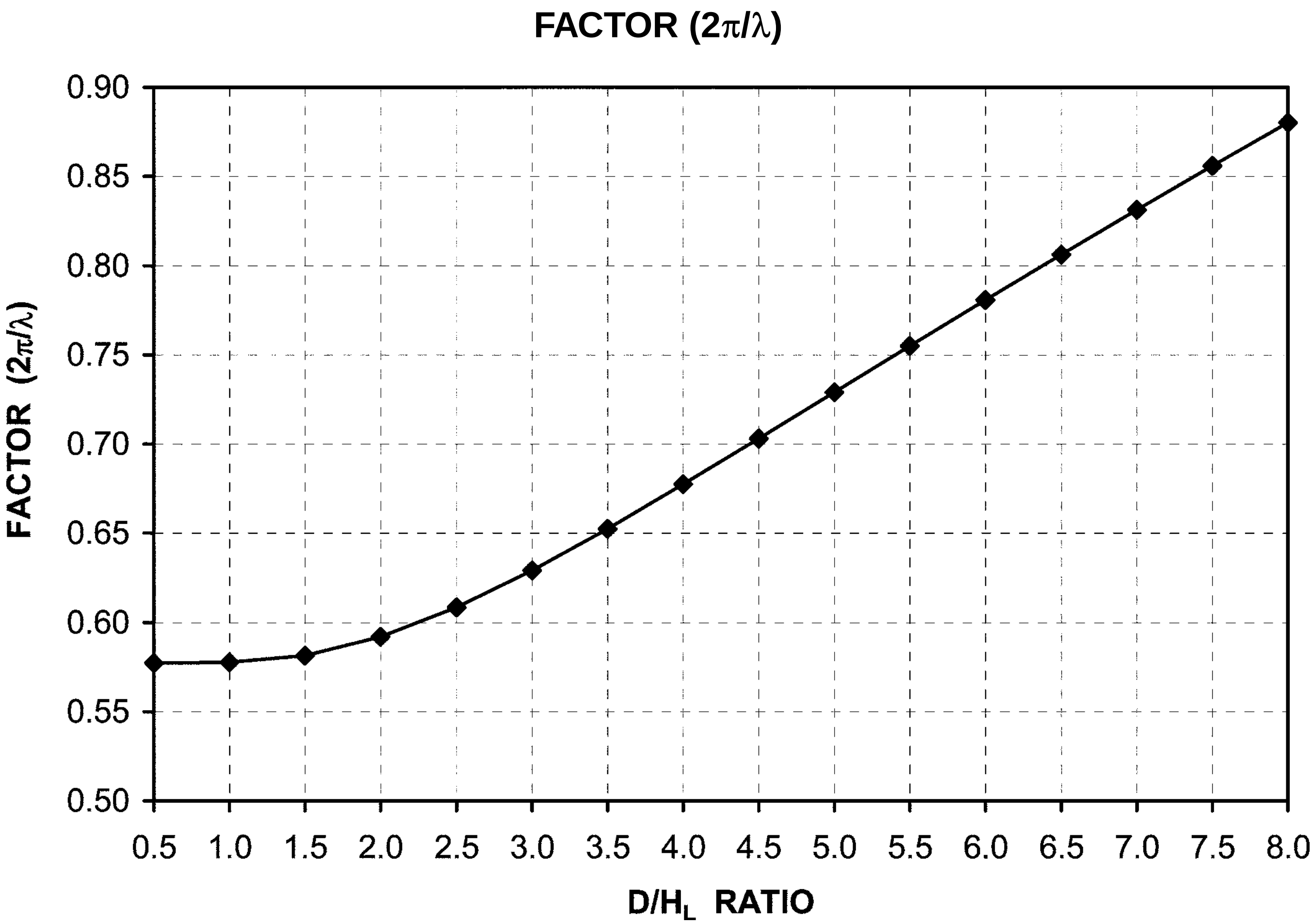


Fig. 9.3.4(b)—Factor $2\pi/\lambda$ *for circular tanks.*

$$\omega_c = \frac{\lambda}{\sqrt{D}} \tag{9-28}$$

$$\lambda = \sqrt{3.68 g \tanh\left(3.68\left(\frac{H_L}{D}\right)\right)} \tag{9-29}$$

$$T_c = \frac{2\pi}{\omega_c} = \left(\frac{2\pi}{\lambda}\right)\sqrt{D} \tag{9-30}$$

CHAPTER 10—COMMENTARY REFERENCES

ACI Committee 350, 2006, "Code Requirements for Environmental Engineering Concrete Structures and Commentary (350-06)," American Concrete Institute, Farmington Hills, Mich., 484 pp.

ACI Committee 371, 1998, "Guide for the Analysis, Design, and Construction of Concrete-Pedestal Water Towers (ACI 371R-98) (Reapproved 2003)," American Concrete Institute, Farmington Hills, Mich., 36 pp.

American Society of Civil Engineers (ASCE), 1981, "Guidelines for the Seismic Design of Oil and Gas Pipeline Systems," Committee on Gas and Liquid Fuel Lifelines of the Technical Council on Lifeline Earthquake Engineering, Section 7.

American Society of Civil Engineering (ASCE), 1984, "Fluid/Structure Interaction During Seismic Excitation," *Report* by Committee on Seismic Analysis.

American Society of Civil Engineers (ASCE), 2005, "Minimum Design Loads for Buildings and Other Structures," ASCE 7-05, Reston, Va.

ANSI/AWWA, 1995a, "AWWA Standard for Wire- and Strand-Wound, Circular, Prestressed Concrete Water Tanks," ANSI/AWWA D110-95.

ANSI/AWWA, 1995b, "AWWA Standard for Circular Prestressed Concrete Water Tanks with Circumferential Tendons," ANSI/AWWA D115-95.

Bertero, V. V., 1995, "Energy Based Design Approach," *Performance-Based Seismic Engineering of Buildings*, SEAOC, Apr., pp. D-1 to D-12.

Building Seismic Safety Council, 1997, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures—Part 1: Provisions (FEMA 302) and Part 2: Commentary (FEMA 303)," Washington, D.C.

Building Seismic Safety Council, 2000, "NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures—Part 1: Provisions (FEMA 368) and Part 2: Commentary (FEMA 369)," Washington, D.C.

Haroun, M. A., 1984, "Stress Analysis of Rectangular Walls Under Seismically Induced Hydrodynamic Loads," *Bulletin of the Seismological Society of America*, V. 74, No. 3, June, pp. 1031-1041.

Haroun, M. A., and Ellaithy, H. M., 1985, "Seismically Induced Fluid Forces on Elevated Tanks," *Journal of Technical Topics in Civil Engineering*, ASCE, V. III, No. 1, Dec., pp. 1-15.

Housner, G. W., 1956, "Limit Design of Structures to Resist Earthquakes," *Proceedings*, World Conference on Earthquake Engineering, University of California, Berkeley, pp. 5-1 to 5-13.

Housner, G. W., 1963, "Dynamic Pressure on Fluid Containers," *Technical Information (TID) Document* 7024, Chapter 6 and Appendix F, U.S. Atomic Energy Commission.

International Code Council, 2003, "International Building Code," 656 pp.

International Conference of Building Officials (ICBO), 1997, "Uniform Building Code," V. 2, Whittier, Calif.

John A. Blume & Associates, 1958, "Report of Testing Program on Earthquake Cable Detail for the Preload Company, Inc.," July.

Medearis, K., and Young, D. H., 1964, "Energy Absorption of Structures Under Cyclic Loading," *Journal of the Structural Division*, ASCE, V. 90, ST1, Feb., pp. 61-89.

National Science Foundation, 1981, "Earthquake Design Criteria for Water Supply & Wastewater Systems," *National Science Foundation Report* NSF/CE52-81079, Sept.

Newmark, N. M., and Hall, W. J., 1982, "Earthquake Spectra and Design," *Earthquake Engineering Research Institute Monograph*.

New Zealand Standard (NZS), 1986, "Code of Practice for Concrete Structures for the Storage of Liquids," NZS 3106, 77 pp.

Scarlat, A. S., 1997, "Design of Soft Stories—A Simplified Energy Approach," *Earthquake Spectra*, V. 13, No. 2, May, pp. 305-315.

Uang, C. M., and Bertero, V. V., 1988, "Use of Energy as a Design Criterion in Earthquake-Resistant Design," *EERC Report* No. UCB/EERC-88, Nov.

Veletsos, S. A., and Shivakumar, P., 1997, "Dynamic Response of Tanks Containing Liquids or Solids," *Computer Analysis and Design of Earthquake-Resistant Structures*, Earthquake Engineering Series V. 3, D. E. Beskos and S. A. Anagnostopoulos, eds., Computational Mechanics Publications, Billerica, Mass., 936 pp.

Notes

COMMENTARY

APPENDIX A—DESIGN METHOD

A.1—General outline of design method

In the absence of a more rigorous method of analysis, the general procedures outlined below may be used to apply the provisions of Chapters 1 through 9.

Basic seismic design parameters:

1. Establish the design depth of the stored liquid H_L, the wall height H_w, and the tank length or diameter, L or D, respectively;

2. From the applicable seismic ground motion map of ASCE 7-05, Chapter 22, obtain the mapped maximum considered earthquake spectral response accelerations at short periods and at 1 second (S_S and S_1, respectively). After selecting the site classification from ASCE 7-05, Table 20.3-1, obtain coefficients F_a and F_v using ASCE 7-05, Tables 11.4-1 and 11.4-2, and calculate S_{DS} and S_{D1} using Eq. (9-35) and (9-36);

3. Select an importance factor I from Table 4.1.1(a);

4. Select the factors R_i and R_c from Table 4.1.1(b) for the type of structure being investigated;

Tank dynamic properties:

5. Calculate the equivalent weight of the tank wall (shell) W_w, roof W_r, and the stored liquid W_L. Also, compute the effective mass coefficient ε;

6. Calculate the effective weight of the impulsive component of the stored liquid W_i, and the convective component W_c using Fig. 9.2.1 for rectangular tanks or Fig. 9.3.1 for circular tanks;

7. Calculate the heights h_w, h_r, h_i, and h_c (EBP) and h_i' and h_c' (IBP) to the center of gravity of the tank wall, roof, impulsive component, and convective component, respectively (Fig. 9.2.2, 9.2.3, 9.3.2, and 9.3.3, or Sections 9.2 and 9.3);

8. Calculate the combined natural frequency of vibration ω_i of the containment structure and the impulsive component of the stored liquid (Eq. (9-9) for rectangular tanks or Eq. (9-23) for circular tank Types 2.1 and 2.2). The impulsive mode will generally fall into the rigid range of the response spectra (that is, the constant spectral acceleration region of the design response spectrum in Fig. R9.4.1) for common sizes of concrete tanks. Thus, if the maximum value of C_i is used (S_{DS}), calculation of the natural frequency and natural period is not required;

9. Calculate the frequency of the vibration ω_c of the convective component of the stored liquid (Eq. (9-12) for rectangular tanks or Eq. (9-28) for circular);

10. Using the frequency values determined in Steps 8 and 9, calculate the corresponding natural periods of vibration T_i and T_c. (Eq. (9-11) and (9-14) for rectangular tanks, or Eq. (9-25), (9-26), and (9-30) for circular tanks);

11. Based on the natural periods determined in Step 10 and the design spectral response acceleration values derived in Step 2, calculate the corresponding seismic response coefficients C_i and C_c (Eq. (9-32), (9-33), (9-37), and (9-38)). Note: Where a site-specific response spectrum is constructed in accordance with Section 4.2.1, C_i and C_c are determined in accordance with Sections 9.5 and R9.5;

Freeboard:

12. Where required, calculate the maximum vertical displacement of liquid surface (wave height) in accordance with Chapter 7. Adjust the wall height if required to meet freeboard requirements;

COMMENTARY

Base shear and overturning moments:

13. Compute the dynamic lateral forces (Eq. (4-1) to (4-4)) and total base shear $\boldsymbol{V}$ (Eq. (4-5));

14. Calculate the bending and overturning moments (Eq. (4-10) and (4-13));

Vertical acceleration:

15. Compute the vertical amplification factor $\boldsymbol{C_t}$ in accordance with Section 9.4.3. For circular tanks, first calculate the natural period of vibration of vertical liquid motion $\boldsymbol{T_v}$ (Eq. (9-31));

16. Calculate the hydrodynamic pressure $\boldsymbol{p_{vy}}$ (Eq. (4-14));

Pressure distribution:

17. Compute the vertical distribution of the force components in accordance with Chapter 5;

Stresses:

18. In rectangular tanks, calculate the stresses in the wall due to the impulsive and convective pressures, depending on the structural system considered (Section 6.1) and the stresses associated with the increase in effective fluid density due to the vertical acceleration. In circular tanks, calculate the hoop stresses due to the impulsive and convective pressures and due to the vertical acceleration (Section 6.2); and

19. Calculate the overall bending stresses due to the overturning moments (from Step 14). Downward pressures on the neoprene bearing pads of free base circular tanks caused by overturning moments should be considered. If uplift develops on the heel side, then anchor cables must be provided.

COMMENTARY

APPENDIX B—ALTERNATIVE METHOD OF ANALYSIS BASED ON 1997 *Uniform Building Code*

B.1—Introduction

B1.1—Scope

The purpose of this appendix is to permit the user to adapt the provisions of ACI 350.3 to the seismic provisions of the 1997 edition of the *Uniform Building Code* (UBC) (ICBO 1997). The differences between the 1997 UBC and the 2003 IBC seismic provisions as used in this Standard are primarily due to differences in the definition of design ground motions and the construction of the corresponding design response spectra as explained below.

NOTE: All section, table, figure and equation references are to 1997 UBC except as otherwise indicated.

B1.2—Design ground motions

This appendix presents an outline of the methodology to be followed when computing the loading side of seismic analysis in accordance with 1997 UBC. In this case, the design ground motions are those with a 10% probability of exceedance in 50 years.

B.2—Notation (not included in Section 1.2 of this standard)

C_a = seismic coefficient, as set forth in Table 16-Q

C_v = seismic coefficient, as set forth in Table 16-R

DL = dead load

E = earthquake load as defined in Section 1630.1

LL = live load

S_A, S_B, S_C, S_D, S_E, S_F = soil profile types as set forth in Table 16-J

N_a = near-source factor used in the determination of C_a in Seismic Zone 4 related to the proximity of the structure to known faults with magnitudes and slip rates as set forth in Tables 16-S and 16-U

N_v = near-source factor used in the determination of C_v in Seismic Zone 4 related to the proximity of the structure to known faults with magnitudes and slip rates as set forth in Tables 16-T and 16-U

T_s = $0.40C_v/C_a$

Z = seismic zone factors as given in Table 16-I

B.3—Loading side, general methodology

1. Select the seismic zone (1 through 4) where the site is located, using the seismic zone map (Fig. 16-2);

2. Using the seismic zone determined in Step 1, find the zone factor Z from Table 16-I;

3. Consulting paragraph 1636.2 and Table 16-J, select the soil profile type designation S_A through S_F that best represents the soil at the site;

4. Using the zone factor Z and the soil profile designation from Steps 2 and 3, find the seismic coefficient C_a from Table 16-Q and seismic coefficient C_v from Table 16-R;

5. If the structure is in Seismic Zone 4, select a Seismic Source Type A, B, or C from Table 16-U, and near-source factors N_a and N_v from Tables 16-S and 16-T, respectively;

COMMENTARY

6. Compute $T_s = \dfrac{C_v}{2.5C_a} = 0.40\dfrac{C_v}{C_a}$;

7. Using the values of C_a, C_v, and T_s, construct a design response spectrum as in Fig. 16-3 and B.1;

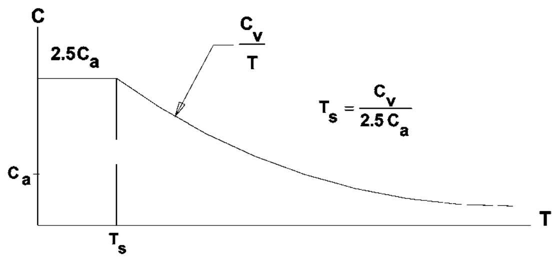


Fig. B.1—Modified UBC 1997 design response spectrum (ICBO 1997).

8. *Seismic response coefficients* **C**$_i$ *and* **C**$_c$—

8.1 **C**$_i$ *(impulsive component)*: Compute period of vibration T_i in accordance with Eq. (9-11) of this Standard for rectangular tanks, and Eq. (9-25) or (9-26) for circular.

(**Note 1:** Section 1634.1.4 specifies the method for determining the fundamental period by referencing 1630.2.2. For liquid-retaining structures, however, the methods in Chapter 9 of this standard should be used.)

Compute seismic response coefficient C_i corresponding to T, using the above design response spectrum as follows

(**Note 2:** Section 1629.8, "Selection of Lateral-force Procedures," allows three options for computing lateral forces, depending on the type of structure being investigated: simplified static, static, or dynamic. For liquid-containing structures, this standard uses the static procedures in accordance with 1629.8.3, modified as indicated in this standard.)

For all seismic zones:

For $T_i \le T_s$

$$C_i = 2.5C_a \tag{B-1}$$

For $T_i > T_s$

$$C_i = C_v/T_i \tag{B-2}$$

In addition, for Seismic Zone 4

$$C_i \ge 1.6ZN_v \tag{B-3}$$

8.2 **C**$_c$ *(convective component)*: Compute period of vibration T_c using Eq. (9-14) of this Standard for rectangular tanks, and (9-30) for circular tanks.

For $T_c \le 1.6/T_s$ seconds

$$C_c = \frac{1.5C_v}{T_c} \le 3.75C_a \tag{B-4}$$

COMMENTARY

For $T_c > 1.6/T_s$ seconds

$$C_c = 6\frac{C_a}{T_c^2} \tag{B-5}$$

9. *Base shears* **V**—

- Compute W_w, W_r, and W_L;
- Compute W_i and W_c using the equations in Sections 9.2 and 9.3 of this Standard;
- Select coefficient R_i and R_c from Table 4.1.1(b) of this Standard.
- Select an importance factor I from Table 4.1.1(a) of this Standard.
- Compute the component parts of the total lateral force, P_w, P_r, P_i, and P_c in accordance with Section 4.1.1 of this Standard as follows.

For all seismic zones

$$P_w = \frac{C_i I}{R_i}\varepsilon W_w \tag{B-6}$$

$$P_r = \frac{C_i I}{R_i}\varepsilon W_r \tag{B-7}$$

$$P_i = \frac{C_i I}{R_i}\varepsilon W_i \tag{B-8}$$

$$P_c = \frac{C_c I}{R_c} W_c \tag{B-9}$$

Equation (B-8) and (B-9) take the following form depending on the period T_i and T_c.

For $T_i \le T_s$

$$P_i = \frac{2.5 C_a I}{R_i} W_i \quad \text{[1997 UBC Eq. (30-5)]}$$

For $T_i > T_s$

$$P_i = \frac{C_v I}{R_i T_i} W_i \ge 0.56 C_a I W_i \quad \text{[1997 UBC Eq. (30-4) and (34-2)]}$$

For $T_c > 1.6/T_s$ seconds

$$P_c = \frac{6 C_a I}{R_c T_c^2} W_c \le \frac{1.5(2.5 C_a) I}{R_c} W_c \tag{B-10}$$

For $T_c \le 1.6/T_s$ seconds

$$P_c = \frac{1.5 C_v I}{R_c T_c} W_c \tag{B-11}$$

COMMENTARY

In addition, for Seismic Zone 4

$$P_i \geq \frac{1.6ZN_vI}{R_i}W_i \quad \text{[1997 UBC Eq. (34-3)]}$$

10. *Total base shear* **V**—The total base shear due to P_w, P_r, P_i, and P_c may be computed by combining these lateral loads using the square root of the sum of the squares method as in Section 4.1.2 of this Standard

$$V = \sqrt{(P_w + P_r + P_i)^2 + P_c^2}$$

11. *Vertical load distribution*—The vertical distribution of the lateral seismic forces may be assumed as shown in Section 5 of this Standard;

12. *Vertical component of ground motion*—Compute the natural period of vibration of the vertical liquid motion T_v in accordance with Section 9.3.4 of this Standard.

Compute seismic response coefficient C_t as follows:

For all seismic zones:

For circular tanks

For $T_v \leq T_s$

$$C_t = C_a \tag{B-12}$$

For $T_v > T_s$

$$C_t = \frac{C_v}{T_v} \tag{B-13}$$

For rectangular tanks, for all periods T_v

$$C_t = C_a \tag{B-14}$$

In addition, for Seismic Zone 4:

For rectangular and circular tanks

$$C_t \geq 1.6ZN_v \tag{B-15}$$

Compute the spectral acceleration $\ddot{u}_v$ as follows

$$\ddot{u}_v = C_tI\frac{b}{R_i} \tag{B-16}$$

13. *Overturning moments*—Compute overturning moments for the lateral loads described above using the procedures of Section 4.1.3 of this Standard. Combine the computed moments using the square root of the sum of the squares method as in the same section.

B.4—Site-specific spectra (Section 1631.2(2))

When a site-specific design response spectrum is available, the coefficients C_i and C_t are replaced by the actual spectral values corresponding to T_i and T_v, respectively, from the 5% damped site-specific spectrum, and C_c is replaced by the actual spectral

COMMENTARY

value corresponding to T_c from the 0.5% damped site-specific spectrum.

If the site-specific response spectrum does not extend into or is not well defined in the T_c range, coefficient C_c may be calculated using Eq. (B-4), with C_v representing the effective site-specific peak ground acceleration expressed as a fraction of the acceleration due to gravity g.

B.5—Resistance side

1. The resistance side of the seismic design, including load combinations and strength reduction factors, may be computed in accordance with the applicable provisions of 1997 UBC (ICBO 1997) or ACI 350-06, Chapter 9; and

2. Where the approved standard defines acceptance criteria in terms of allowable stresses (as opposed to strengths), the design seismic forces obtained from this appendix shall be reduced by a factor of 1.4 for use with allowable stresses, and allowable stress increases used in the approved standard are permitted. When such a standard is used, the following load combinations are permitted to be used for design instead of the ASCE 7-05 load factor combinations (Haroun and Ellaithy 1985).

$$DL + LL + E/1.4$$

B.6—Freeboard

$$d_{max} = C_c I\left(\frac{L \text{ or } D}{2}\right) \tag{B-17}$$

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