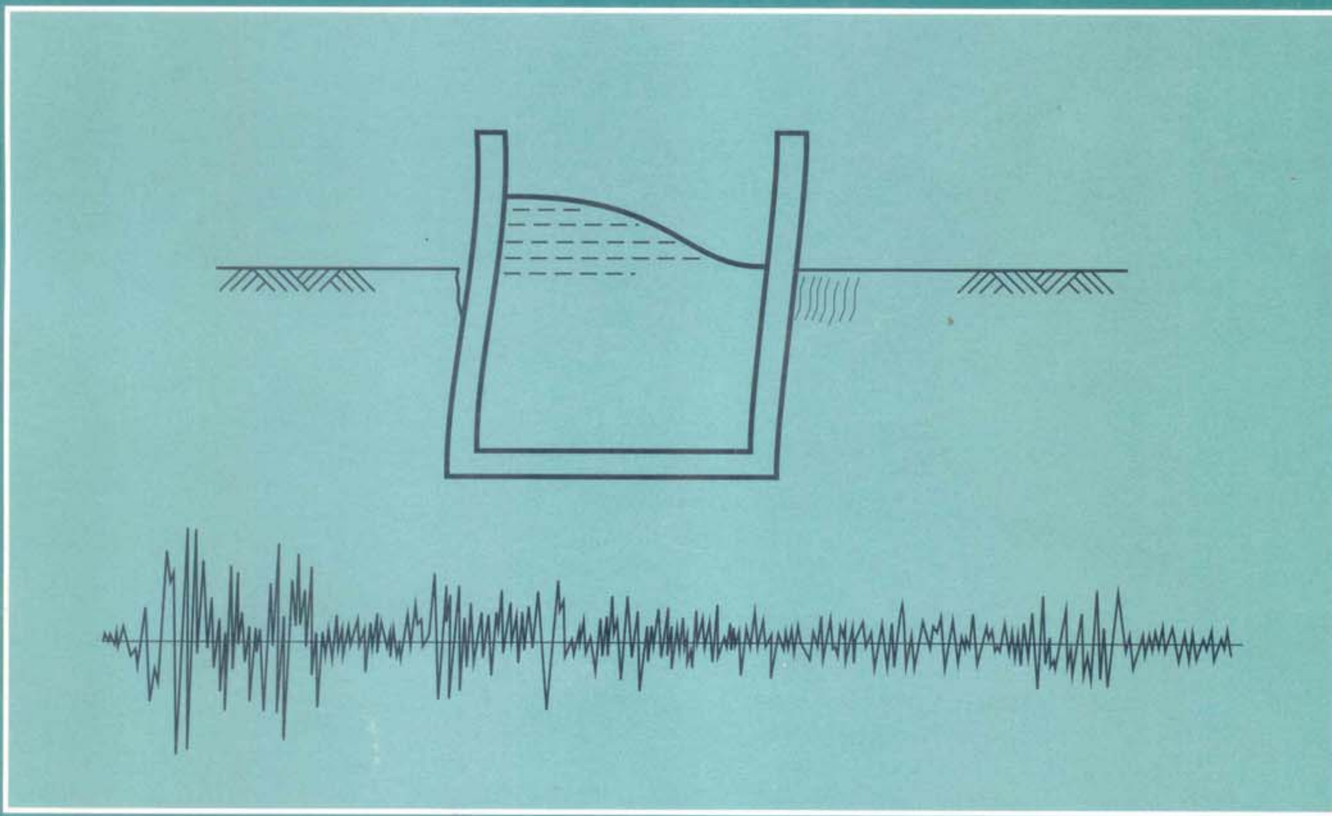
Design of Liquid-Containing Concrete Structures for Earthquake Forces

by Javeed A. Munshi



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Construction Technology Laboratories, Inc.

PORTLAND CEMENT ASSOCIATION

An organization of cement manufacturers to improve and extend the uses of portland cement and concrete through market development, engineering, research, education and public affairs work.

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Contents

CHAPTER1

Introduction

1.1 GENERAL

This publication provides requirements and guidelines for the design and detailing of liquid-containing structures for earthquake forces using the IBC 2000, UBC 1997, UBC 1994, BOCA 1996 and SBC 1997 model codes. Note that although these codes themselves do not contain specific provisions for detailed seismic analysis and design of liquid-containing structures, they do allow use of consensus industry standards. The report of Committee 350-01 on the *Code Requirements for Environmental Engineering Concrete Structures* of the American Concrete Institute meets this requirement of being a nationally recognized consensus standard applicable to liquid-containing and other environmental structures. The committee recently published *Seismic Design of Liquid-Containing Concrete Structures (ACI 350.3-01) and Commentary (ACI 350.3R-01)*, which gives detailed procedures for seismic analysis and design of liquid-containing structures. Furthermore, Chapter 21 of ACI 350-01 gives provisions for seismic design of liquid-containing structures much in the same manner as Chapter 21 of ACI 318 does for building structures.

Note that ACI 350-01 refers to ACI 318-95 while ACI 350.3-01 is compatible with UBC 1994 service-level earthquake design methodology. The provisions of ACI 350.3-01 are not presently compatible with IBC 2000, UBC 1997, BOCA 1996 and SBC 1997 for two reasons: (a) All of these model codes use strength-level earthquake forces, and (b) Some of the model codes are based on more recent (post-1977) seismic hazard analysis and seismic zoning, which have not been incorporated in ACI 350.3. Therefore, ACI 350.3-01 in its current form can not be directly used with these building codes.

This publication bridges the gap between ACI 350.3-01 and the model codes indicated above which use strength-level earthquake forces. The concepts of ACI 350-01 and ACI 350.3-01 have been extended for use with the IBC 2000, UBC 1997, BOCA 1996 and SBC 1997 for the design of liquid-containing structures. An effort has been made to interpret and extrapolate the concepts and provisions of ACI 350.3-01 to make them compatible with these codes.

Several challenges were encountered during this process because of incompatibilities and insufficient information pertaining to load combinations involving earthquake and fluid pressure, response modification factors corresponding to impulsive and convective motion of liquid, effects of dynamic earth and ground water pressure and the manner in which they should be combined with other loads, and last but not least, complexity of structural response and boundary conditions. Since many of these issues are complex and currently being researched and debated, the interpretations given in this publication are by no means final. ACI Committee 350 has identified some of these issues to be taken up as future business. This publication will be updated when either more information or better interpretations of these issues become available through the consensus committee efforts.

1.2 SCOPE

This publication is meant as a guide for the design and detailing of concrete liquid-containing structures for earthquake forces according to the model building codes. It covers rectangular and circular tanks with non-flexible and flexible wall-to-base slab connections.

Chapter 2 provides basic information on the different types of tanks and their wall-to-base slab connections.

Chapter 3 summarizes the required earthquake design loads as prescribed in the model codes noted above. It also contains the applicable code-prescribed load combinations for design of concrete structures. A discussion on various load combinations involving earthquake and fluid pressure loads applicable to liquid-containing concrete structures is also presented.

Chapter 4 describes modeling and analysis methods using different model codes. The concepts of ACI 350.3-01 are integrated with the loading provisions of the IBC 2000, UBC 1997, UBC 1994, BOCA 1996, and SBC 1997. This chapter also gives equations for determining the period of the structure based on the boundary conditions of its walls. Charts and design aids for determining the impulsive and convective weights along with the height are also given.

Chapter 5 contains the design recommendations for various components of the liquid-containing structure, including foundations, immersed elements and other components subjected to dynamic loads. Procedures for design of both rectangular and circular tank walls are included.

Chapter 6 summarizes the detailing requirements for walls based on ACI 318-99. The overriding provisions of ACI 350-01 are also noted wherever applicable.

Chapter 7 illustrates the design and detailing of a rectangular concrete tank located in a region of high seismicity per the IBC 2000. The base shear is also computed for the UBC 1997, UBC 1994, BOCA 1996 and SBC 1997 codes.

Chapter 8 illustrates the design and detailing of a nonprestressed circular concrete tank located in a region of high seismicity, per the IBC 2000.

CHAPTER 2

General

2.1 TYPES OF LIQUID-CONTAINING STRUCTURES

i. **Rectangular**
   1. **Fixed Base** (Fig. 2-1(a))
   2. **Hinged Base** (Fig. 2-1(b))

ii. **Circular without Prestressing**
   1. **Fixed Base** (Fig. 2-1(a))
   2. **Hinged Base** (Fig. 2-1(b))

iii. **Circular with Prestressing**
   1. **Fixed Base** (Fig. 2-1(a))
   2. **Hinged Base** (Fig. 2-1(b))
   3. **Flexible Base**
      a. Anchored (Fig. 2-2(a))
      b. Unanchored, Contained (Fig. 2-2(b))
      c. Unanchored, Uncontained (Fig. 2-2(c))

Liquid-containing structures essentially fall into two categories of behavior based on their wall-to-footing connection: the non-sliding or the rigid base (Fig. 2-1) and the flexible base (Fig. 2-2). The non-sliding base typically uses a fixed or hinged wall-to-footing connection. The flexible base typically uses a base pad between the wall and the footing and allows varying degrees and types of movement depending upon whether the wall is anchored, unanchored contained or unanchored uncontained in the footing (Fig. 2-2). This type of connection is only used for circular prestressed tanks. The type of base connection is likely to influence the seismic response of a liquid-containing structure and its effect should be properly included in modeling, design and detailing.

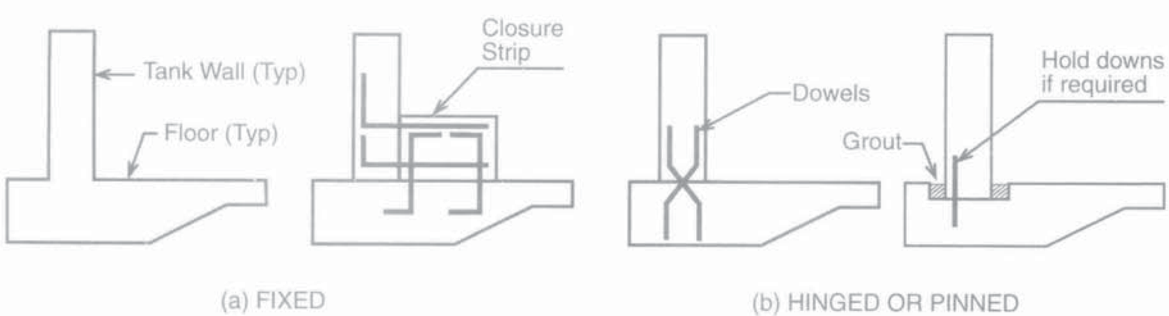


Figure 2-1. Nonflexible Base Connections

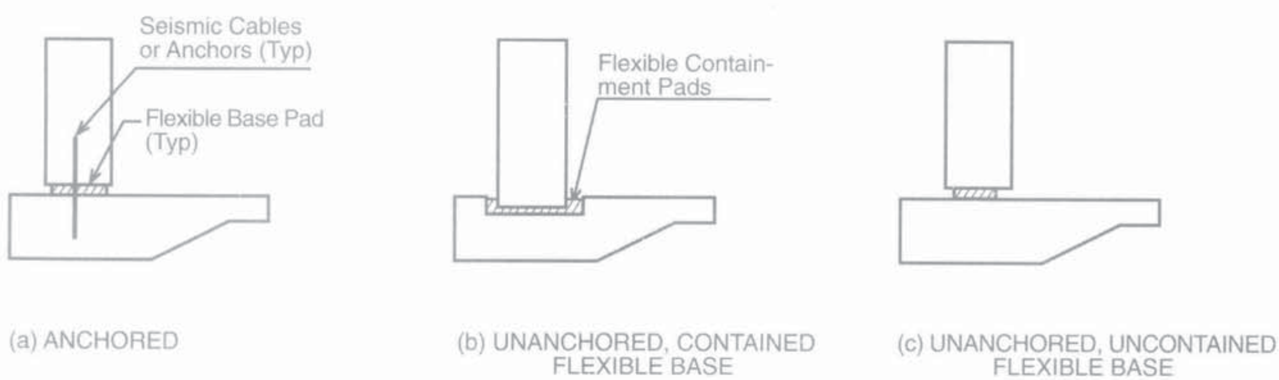


Figure 2-2. Flexible Base Connections

Non-sliding Base. Tanks that have a fixed or hinged connection between the walls and the foundation are essentially non-sliding type. Steel reinforcement or keying action ensures the non-sliding behavior, as shown in Fig. 2-1.

Anchored Flexible Base. Tanks with flexible base that use some kind of anchorage between the wall and the footing allow radial movement but restrict the tangential movement at the base of the structure. Typically, anchorage is achieved with strand cables embedded in the wall and the footing (Fig. 2-2(a)). Compressible sleeves are used over anchor cables at the base joint to allow radial wall movement.

Unanchored Contained Flexible Base. These tanks use an unanchored wall contained by a concrete curb as shown in Fig. 2-2(b). This type of connection allows limited radial and tangential movement.

Unanchored Uncontained Flexible Base. These allow an unlimited radial and tangential movement of the joint since no anchorage or containment of the walls is involved (Fig. 2-2(c)). This type of tank is not permitted in regions of high seismicity (UBC zones 3 and 4) for obvious reasons of potentially uncontrolled movement during a seismic event.

CHAPTER 3
Earthquake Design Loads

3.1 IBC 2000 METHOD

3.1.1 General

Design of liquid-containing structures falls under Section 1622 of the International Building Code[3-1] (IBC 2000) for non-building structures. This section contains more elaborate provisions both for elevated and on/above ground tanks as compared to the 1997 Uniform Building Code[3-2] (UBC '97), the 1997 Standard Building Code[3-3] (SBC '97) and the 1996 BOCA National Building Code[3-4] (BOCA '96).

Section 1622.2 of IBC 2000 indicates that when calculating the seismic forces, the normal operating contents should be included for tanks. The seismic weight W should also include snow and ice loads when these constitute more than 25% of W.

The fundamental period can be determined by Section 1617.4.2 of IBC or by using any other rational method such as given in Chapter 4 of this publication. The displacement, drift and the P-Δ effects are generally not significant for at/or below grade tanks.

The seismic coefficients R, C_D and Ω_o are given in IBC Table 1622.2.5 (1), based on the structure type. The importance factors are given based on the seismic use group and the hazard/function criteria in IBC Table 1622.2.5 (2).

The seismic effect E to be used in load combinations shall be determined using Section 1617.1 reproduced below (Section 3.1.4). The redundancy/reliability factor (ρ) shall be taken as 1. The base shear is computed using Section 1617.4.1.

3.1.2 Base Shear

Section 1622.2.5 stipulates that the minimum seismic base shear shall not be less than that computed in accordance with the requirements of 1617.4.1. The procedure for computing the base shear is as follows:

$$V = C_S W$$

$$C_S = \frac{S_{DS} I}{R} \le \frac{S_{D1} I}{RT}$$

$$C_S \ge 0.14 S_{DS} I \quad \text{Eq. 16-75 (Section 1622.2.5)}$$

$$C_S \ge \frac{0.8 S_1 I}{R} \quad \text{Eq. 16-76 (Section 1622.2.5)}$$

where,

I = Importance factor (Table 1622.2.5 (2))

R = Seismic Coefficient (Table 1622.2.5 (1))

W = effective seismic weight of the structure per 1617.4

S_{DS} = the design spectral response acceleration at short period obtained from Section 1615.1.3.

S_{D1} = the design spectral response acceleration at 1 second period obtained from Section 1615.1.3.

$S_{DS} = 2/3\, S_{MS}$

$S_{D1} = 2/3\, S_{M1}$

S_{MS} = maximum considered earthquake spectral response acceleration at short period (1615.1.2).

S_{M1} = maximum considered earthquake spectral response acceleration at 1 second period (1615.1.2).

$S_{MS} = F_a S_S$

$S_{M1} = F_v S_1$

F_a, F_v = site coefficients defined in Table 1615.1.2. Tables 1622.2.5 (1) and 1622.2.5 (2) give the values for F_a and F_v corresponding to the site class. The site class can be determined by using Table 1615.1.1.

S_S, S_1 = the mapped spectral accelerations for short period and 1 second periods, respectively, as determined in Section 1615.1.

T = fundamental period (See Sections 1617.4.2.1 and Chapter 4)

3.1.3 Rigid Structures

The liquid-containing structures that have a fundamental period, T, less than 0.06 s, including their anchorages, shall be designed for the lateral force obtained from the following (1622.2.6):

$$V = 0.3 S_{DS} W I$$

where S_{DS}, W and I are as defined previously.

3.1.4 Seismic Load Effect

The earthquake induced force from the combined horizontal and vertical acceleration effects is determined in accordance with Sections 1622.4.1 and 1617.4.1 as follows:

$$E = Q_E \pm 0.2S_{DS}D$$

Q_E = effect of horizontal seismic forces in the element due to base shear V

D = effect of dead load

Exception: where $S_s \leq 0.15g$ and $S_1 \leq 0.04g$, the structure shall be designed for Seismic Design Catagory A.

3.1.5 Above-Grade Storage Tanks

The liquid-containing structures mounted above grade in structures, the attachments, supports and the tank shall be designed to meet the force requirements of Section 1621.1.4, with R_p equal to R specified in Section 1622. The weight of the storage tank (W_p) shall include the weight of the tank structure and appurtenances and the operating weight of the contents at maximum rated capacity.

Further, when the sloshing period of the stored liquid is within 70% to 150% of the fundamental period of the supporting structure, the effects of sloshing shall be included in the design of tank and its supporting structure.

3.1.6 At-Grade Storage Tanks

According to Section 1622.4.3.2, storage tanks mounted at the base shall be designed to meet the design requirements of Section 1622. In addition, for sites where S_{DS} is greater than 0.60, flat bottom tanks designed with I_p greater than 1.0 and tanks greater than 20 ft in diameter and tanks that have a height-to-diameter ratio greater than 1.0 are required to meet certain conditions in 1622.4.3.2.

3.2 UBC '97 METHOD

3.2.1 Rigid Structures

In the Uniform Building Code, the design of tanks with supported bottom falls under Section 1634.4 for nonbuilding structures. This section assumes that tanks with supported bottoms or flat-bottom tanks founded at or below grade are inherently rigid. Accordingly, such structures are to be designed to resist seismic forces evaluated using the procedure given in Section 1634.3 for rigid structures. Section 1634.4 also specifies that the entire weight of the tank including its contents should be used in the analysis. The procedure is outlined as follows:

Design seismic lateral force F_p,

$$F_p = 0.7C_a I_p W_p$$

where C_a = seismic coefficient (Table 16-Q of UBC '97)
I_p = importance factor (Table 16-K of UBC '97)
W_p = weight of tank and contained liquid

3.2.2 Alternate Methods

Section 1634.4 allows the following two methods as an alternative to Section 3.2.1:

1. A response spectrum analysis that includes consideration of actual ground motion anticipated at the site and the inertial effects of the contained liquid.

2. A design basis for the particular type of tank by an approved national standard, provided that the seismic zones and occupancy categories are in conformance with provisions of Sections 1629.4 and 1629.2, respectively.

3.3 BOCA National Building Code and the Standard Building Code (SBC) Method

The BOCA and the SBC Codes do not give specific provisions for design of liquid-containing structures. Section 1610.1 of the BOCA National Building Code and Section 1607.1.1 of the Standard Building Code indicate that provisions of ASCE 7[3-5] may be used to design such structures. This method is similar to the one given in UBC 1997. These codes further specify that special structures shall be designed for earthquake loads utilizing an approved substantiated analysis.

3.4 ACI 350.3 METHOD

Note that both the IBC 2000 (1622.2.5) and the UBC 1997 (1634.4) provisions allow the use of an alternate design procedure from a nationally approved standard. The provisions of ACI 350.3[3-6] can be used as an alternate design procedure satisfying the criteria of a nationally approved standard. Various concepts of ACI 350.3 have been included in Chapter 4. Note that the provisions of ACI 350.3 are compatible with the UBC 1994 that involves service-level earthquake forces. The IBC 2000, UBC 1997, BOCA 1996 and SBC 1997 use strength-level earthquake forces.

3.5 CONCRETE-PEDESTAL WATER TOWERS

ACI Committee Report 371-98[3-7] presents detailed recommendations for materials, analysis, design and construction of concrete-pedestal elevated water storage towers. These structures generally consist of steel storage tanks supported by a cylindrical reinforced concrete pedestal. The report contains detailed recommendations for seismic design of such pedestals. It is recommended that the user refer to this document for design of concrete-pedestals.

3.6 BURIED STRUCTURES

Section 1622.4.8 of the IBC indicates that structures buried underground shall be designed for seismic forces determined by a substantiated analysis using standards approved by the building official. However, no guidelines exist for design of such structures in either the IBC or UBC. The analysis and design of buried structures should include the effect of dynamic earth pressure. Research[3-8] has, however, shown that seismic pressures do not control design unless the peak ground acceleration exceeds about 0.3g, where g is the acceleration due to gravity. Thus, the design of buried tanks located in low to moderate seismic risk areas is likely to be governed by static loading. Also, the effect of soil nonlinearities due to local soil failure seems to have little effect on the predictions of the seismic response of buried structures[3-8].

3.7 LOAD COMBINATIONS

3.7.1 General

The load combinations of IBC 2000 that are applicable to the design of general building structures are given in Table 3-1. The load combinations which include seismic effects are based on ASCE 7-98,[3-5] while the non-seismic load combinations are based on ACI 318-99. Table 3-2 gives a comparison of load combinations in the 1994 and 1997 UBC. The load combinations of 1996 BOCA and 1997 SBC are given in Table 3-3.

The building codes do not give specific load combinations that can be directly used for design of liquid-containing structures for earthquake loading. Section 1605.2.2 of IBC 2000 indicates that fluid pressure (F) should be added in the prescribed load combinations in accordance with

Table 3-1 Load Combinations for Building Structures Per IBC 2000

Code Section	Loads†	Required Strength	Code Eq. No.
ACI 9.2.1	Dead (D) & Live (L)	$U = 1.4D + 1.7L$	(9-1)
ACI 9.2.2	Dead, Live & Wind (W)	(i) $U = 1.4D + 1.7L$ (ii) $U = 0.75\ (1.4D + 1.7L + 1.7W)$ (iii) $U = 0.9D + 1.3W$	(9-1) (9-2) (9-3)
ACI 9.2.3 IBC 1605.2 IBC 1605.2	Dead, Live & Earthquake (E)	(i) $U = 1.4D + 1.7L$ (ii) $U = 1.2D + 1.0E + (f_1L + f_2S)$ (iii) $U = 0.9D + 1.0E$	(9-1) (16-5) (16-6)
ACI 9.2.4	Dead, Live & Earth and Groundwater Pressure (H)**	(i) $U = 1.4D + 1.7L$ (ii) $U = 1.4D + 1.7L + 1.7H$ (iii) $U = 0.9D + 1.7H$ where D or L reduces H	(9-1) (9-4)
ACI 9.2.5	Dead, Live & Fluid Pressure (F)***	(i) $U = 1.4D + 1.7L$ (ii) $U = 1.4D + 1.7L + 1.4F$ (iii) $U = 0.9D + 1.4F$ where U or L reduces F	(9-1)
ACI 9.2.7	Dead, Live and Effects from Differential Settlement, Creep, Shrinkage, Expansion of Shrinkage-Compensating Concrete, or Temperature (T)	(i) $U = 1.4D + 1.7L$ (ii) $U = 0.75\ (1.4D + 1.4T + 1.7L)$ (iii) $U = 1.4\ (D + T)$	(9-1) (9-5) (9-6)

† *D, L, W, H, F, and T represent the designated service loads or their corresponding effects such as moments, shears, axial forces, torsion, etc. Note: E is a strength-level earthquake force.*
** *Weight and pressure of soil and water in soil. (Groundwater pressure is to be considered part of earth pressure with a 1.7 load factor.)*
*** *Weight and pressure of fluids with well-defined densities and controllable maximum heights.*

Table 3-2 Comparison of Load Combinations in the 1994 and 1997 UBC for Building Structures

1994 UBC		1997 UBC	
Eq. No.	Load Combination (1909.2, 1921.2.7)	Eq. No.	Load Combination (1612.2, 1909.2)
(9-1)	$1.4D + 1.7L$	(9-1)	$1.4D + 1.7L$
(9-2)	$0.75\ (1.4D + 1.7L + 1.7W)$	(9-2)	$0.75\ (1.4D + 1.7L + 1.7W)$
	$1.4\ (D + L + E)^*$	(9-3)	$0.9D + 1.3W$
	$0.75\ (1.4D + 1.7L + 1.87E)^\dagger$	(12-5)	$1.1\ [1.2D + 1.0E + (f_1L + f_2S)]$
(9-3)	$0.9D + 1.3W$	(12-6)	$1.1\ (0.9D + 1.0E)$
	$0.9D + 1.4E^*$		
	$0.9D + 1.43E^\dagger$		
	Special Load Combination (1628.7.2)		Special Load Combination (1612.4)
	$1.0D + 0.8L + 3\ (R_W/8)\ E$	(12-17)	$1.2D + f_1L + 1.0E_m$
	$0.85D + 3(R_W/8)\ E$	(12-18)	$0.9D + 1.0E_m$

R_W = response modification factor (Table 16-N, 1994 UBC)
$E = \rho E_h + E_v$ Eq. (30-1), UBC 1997
$E_m = \Omega_o E_h$ Eq. (30-2), UBC 1997
E_h = earthquake load due to the base shear (V)
E_v = earthquake effect due to vertical component of ground motion
ρ = reliability/redundancy factor
Ω_o = seismic force amplification factor (Table 16-N)
f_1, f_2 = factors defined in 1612.21
**Seismic Zones 3 and 4*
†Seismic Zones 2 and lower

Table 3-3 Load Combinations Specified in BOCA and SBC for Building Structures*

Equation No./Code	BOCA 1996**	Equation No./Code	SBC 1997***
(9-1) ACI 318	$1.4D + 1.7L$	(9-1) ACI 318	$1.4D + 1.7L$
(9-2) ACI 318	$0.75\ (1.4D + 1.7L \pm 1.7W)$	(9-2) ACI 318	$0.75\ (1.4D + 1.7L \pm 1.7W)$
(9-3) ACI 318	$0.9D \pm 1.3W$	(9-3) ACI 318	$0.9D \pm 1.3W$
(5) ASCE 7	$(1.2 + 0.5A_V)\ D + 0.5L \pm 1.0Q_E$	(1) SBC	$(1.1 + 0.5A_V)\ D + L \pm Q_E$
(6) ASCE 7	$(0.9 - 0.5A_V)\ D \pm 1.0Q_E$	(2) SBC	$(0.9 - 0.5A_V)\ D \pm Q_E$
(5)† ASCE 7	$(1.2 + 0.5A_V)\ D + 0.5L \pm (2R/5)\ Q_E$	(4)† SBC	$(1.1 + 0.5A_V)\ D + L \pm (2R/5)\ Q_E$
(6)†† ASCE 7	$(0.9 - 0.5A_V)\ D \pm (2R/5)\ Q_E$	(3)†† SBC	$(0.9 - 0.5A_V)\ D \pm (2R/5)\ Q_E$

* D = *Effect of dead loads*
L = *Effect of live loads*
W = *Effect of wind loads*
Q_E = *Effect of horizontal seismic forces*
A_V = *Seismic coefficient representing effective peak velocity-related acceleration (1610.1.3 of BOCA, 1607.1.5 of SBC)*
R = *Response modification factor (1610.3.3 of BOCA, 1607.3.3 of SBC)*
** *See 1613.0, 1610.3.7, Sect. 9.2 of ACI 318, and Sect. 2.3.2 of ASCE 7-95.*
*** *See 1609.2 and Sect. 9.2 of ACI 318-99.*
† *Used for axial compression in columns supporting discontinuous lateral force resisting elements, where $(2R/5) \geq 1.0$.*
†† *Used for horizontal prestressed members in buildings assigned to SPC D or where $(2R/5) \geq 1.0$.*

Section 2.3.2 of ASCE 7-98. However, Section 2.3.2 of ASCE 7-98 does not specify any load combination involving the effects of fluid pressure (F) and the earthquake force (E). Therefore, the manner in which F and E should be combined and the load factors associated with them remain unclear.

Section 9.2.5 of ACI 318 indicates that fluid pressure (F) should be added to all load combinations that involve live load (L). Section 9.2.5 of ACI 350 overrides the ACI 318 requirement by clarifying that F should be added to all governing load combinations, so that the effect of L, W or E does not reduce the effect of F. Based on this, load combinations consistent with ACI 318 and ACI 350, excluding the effect of environmental durability discussed in Section 3.7.2 below, can be formulated for use with different codes as follows:

For IBC 2000:

$U = 1.2D + 1.0E + 1.2F + (f_1L + f_2S)$	Eq. (16-5), IBC 2000
$U = 0.9D \pm 1.0E + 1.2F$	Eq. (16-6), IBC 2000
$U = 0.75(1.4D + 1.7L + 1.4F + 1.87E/1.4)$	Eq. (9-2), ACI 318

For UBC 1997:

$U = 1.1[1.2D + 1.0E + 1.3F + (f_1L + f_2S)]$	Eq. (12-5), UBC 1997*
$U = 1.1[0.9D \pm 1.0E + 1.3F]$	Eq. (12-6), UBC 1997*
$U = 0.75(1.4D + 1.7L + 1.4F + 1.87E/1.4)$	Eq. (9-2), ACI 318

*See Section 1612.2.2

For UBC 1994:

$U = 1.4\,(D + L + F + E)$	Eq. (9-2), UBC 1994
$U = 0.9D \pm 1.4E + 1.4F$	Eq. (9-3), UBC 1994
$U = 0.75(1.4D + 1.7L + 1.4F + 1.87E)$	Eq. (9-2), ACI 318*

*Zones 1 and 2

For SBC 1997:

$U = (1.1 + 0.5A_v)D + L + 1.2F \pm Q_E$	Eq. (1), SBC 1997
$U = (0.9 - 0.5A_v)D \pm Q_E + 1.2F$	Eq. (2) SBC 2000
$U = 0.75(1.4D + 1.7L + 1.4F + 1.87E/1.4)$	Eq. (9-2), ACI 318

For BOCA 1996:

$U = (1.2 + 0.5A_v)D + 0.5L + 1.2F \pm Q_E$	Eq. (5), ASCE 7
$U = (0.9 - 0.5A_v)D \pm Q_E + 1.2F$	Eq. (6) ASCE 7
$U = 0.75(1.4D + 1.7L + 1.4F + 1.87E/1.4)$	Eq. (9-2), ACI 318

The IBC 2000, the UBC 1997, the BOCA 1996 and SBC 1997 use the strength-level earthquake force while the UBC 1994 and ACI 318 use the service-level earthquake force.

Note that in the absence of clearly defined load combinations, the load combinations given above are the best interpretations of what is currently in the building codes as it applies to liquid-containing structures. The designer should carefully investigate the load combinations that apply to his/her situation.

3.7.2 Consideration for Environmental Durability

ACI 350[3-9] requires the following two modifications to the load combinations. The second modification is not applicable to those load combinations that include seismic effects.

Modification 1—The load factor to be used for lateral liquid pressure, F, is 1.7 rather than 1.4. This value of 1.7 may be overconservative for some tanks, since they are filled to the top only during leak testing or because of accidental overflow. Since leak testing usually occurs only once and since most tanks are equipped with overflow pipes, some designers have considered using the load factor of 1.4 in an attempt to reduce the amount of required steel, which would result in less shrinkage restraint. However, this publication suggests that tank designs meet ACI 350 and, therefore, recommends the use of a load factor of 1.7 with F.

Modification 2—The members must be designed to meet the required strength, U, increased by a multiplier called the environmental durability factor (EDF). The EDF will increase the design loads to provide a more conservative design with less cracking. The increased required strength is given by:

Required strength = EDF x U

where the EDF equals:

1.3 for flexural reinforcement

1.65 for direct tension reinforcement

1.3 for shear beyond that of the capacity provided by the concrete

For example, the strength equations based on ACI 318 are given as follows:

1. Flexural Reinforcement

   Req'd strength $\geq 1.3\,U$

   $$\phi M_n \geq 1.3\,(1.4M_D + 1.7M_L + 1.7M_F)$$

2. Direct Tension Reinforcement

   Req'd Strength $\geq 1.65\,U$

   $$\geq 1.65\,(1.4T_D + 1.7T_L + 1.7T_F)$$

3. Stirrup Reinforcement

   $$\phi V_S \geq 1.3\,(V_U - \phi V_C)$$

4. Concrete Shear and Compression

   Req'd Strength $> 1.0U$

No increase is required in load factors for concrete shear, bond, or compression strength, so that proportioning member depths or thickness will be unchanged. For flexure, the proposed increase in load factors results in a maximum load factor of 1.3 times 1.7 = 2.21 for normal live and water and earth load and a minimum load factor of 1.3 times 1.4 = 1.82 for all dead load. In conjunction with ϕ-factors prescribed in ACI 318, these new load factors result in flexural service load stresses in the reinforcement between 24 and 29 ksi, consistent with allowable stresses for working stress design in the current report by ACI Committee 350.

3.8 REFERENCES

3-1. *International Building Code*, International Code Council, Falls Church, VA, March 2000.

3-2. *Uniform Building Code*, International Conference of Building Officials (ICBO), Whittier, CA, 1997.

3-3. *Standard Building Code*, Southern Building Code Congress International, Birmingham, AL, 1997.

3-4. *The BOCA National Building Code*, Building Officials and Code Administrators International, Country Club Hills, IL, 1996.

3-5. *Minimum Design Loads for Buildings and Other Structures*, ASCE 7-98 and ASCE 7-95 American Society for Civil Engineers, New York.

3-6. *Seismic Design of Liquid-Containing Concrete Structures (ACI 350.3-01) and Commentary (ACI 350.3R-01)*, ACI Committee 350, American Concrete Institute, Farmington Hills, MI, 2001.

3-7. *Guide for the Analysis, Design and Construction of Concrete-Pedestal Water Towers*, Reported by ACI Committee 371, American Concrete Institute, Farmington Hills, MI, 1998.

3-8. Miller, C. A. and Costantino, C. J., "*Seismic Induced Earth Pressures in Buried Vaults*", American Society of Mechanical Engineers (ASME), PVP-Vol. 271, 1994, pp. 3-11.

3-9. *Code Requirements for Environmental Engineering Concrete Structures (ACI 350-01) and Commentary (ACI 350R-01), Committee 350*, American Concrete Institute, Farmington Hills, MI, 2001.

CHAPTER 4

Modeling and Analysis

This chapter gives detailed procedures for computation of seismic base shear and overturning moment of liquid-containing structures for different model codes. The spectrum approach of seismic design and the effects of vertical accelerations and earth pressure are also included.

4.1 MODELING

The liquid-containing structures are modeled using Housner's method[4-1,4-2]. This method essentially assumes that hydrodynamic effects due to seismic loading can be evaluated approximately as the sum of the following two parts:

1. Impulsive part, which represents the portion of the liquid which moves in unison with the structure and,
2. Convective part, which represents the effect of the sloshing action of the liquid.

Figure 4-1 shows the typical schematic of a rectangular tank with length L, width B and height of liquid H_L. A similar schematic is shown for a circular tank of diameter D.

The impulsive weight of liquid (W_i) is assumed to be rigidly attached to the structure at height h_i, while the convective weight of liquid (W_c) is attached to the structure by springs of finite stiffness and damping at height h_c as shown in Fig. 4-2. For concrete tanks with rigid walls and roof, this results in a two degree-of-freedom system (Fig. 4-2). Both the impulsive and the convective components have a period associated with them that are generally far apart. The total approximate response of the system can be estimated by the square root of the sum of squares (SRSS) combination of the responses associated with the two periods. Figure 4-3 shows the various mode shapes of rectangular and circular liquid-containing structures.

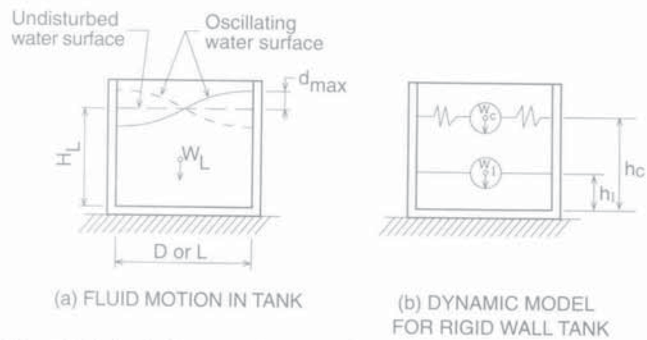


Fig. 4-2 Tank Dynamics

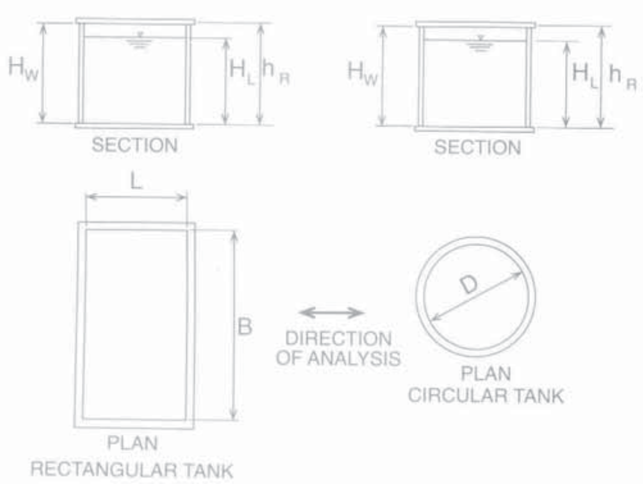


Fig. 4-1 Schematic of Rectangular and Circular Tank

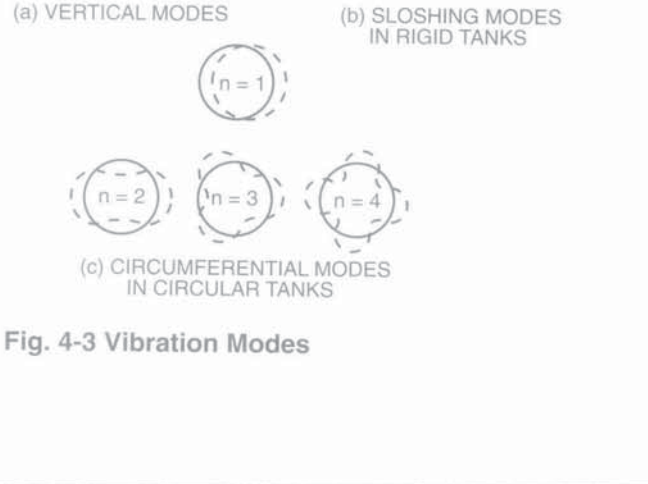


Fig. 4-3 Vibration Modes

4.2 IBC 2000 METHOD

The base shear equation given in Chapter 3 is modified to include the impulsive and the convective components for liquid-containing structures as shown below.

4.2.1 Base Shear

$$V_I = C_{SI}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

$$V_C = C_{SC}W_C \qquad \text{Convective}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2}$

$$C_{SI} = \frac{S_{DS}I}{R} \le \frac{S_{D1}I}{RT_I}$$

$$C_{SC} = \frac{S_{DS}I}{R} \le \frac{S_{D1}I}{RT_C}$$

Per Section 1622.2.5, $C_{SI} \ge 0.14S_{DS}I$

$$C_{SI} \ge \frac{0.8S_1I}{R}$$

The quantities W_W, W_R, W_I and W_C represent the wall weight, roof weight, impulsive weight of the fluid and the convective weight of the fluid, respectively. The impulsive weight W_I and convective weight W_C can be determined as a fraction of the total liquid weight from Fig. 4-4a and 4-4b for rectangular and circular tanks, respectively. The height at which the impulsive and convective weights are assumed to act can be determined from Figs. 4-5a and 4-5b for rectangular and circular tanks, respectively.

The quantities S_{DS} and S_{D1} are determined as described in Section 3.1.2, where I = importance factor (IBC Table 1622.2.5 (2)), R = response modification factor (IBC Table 1622.2.5 (1)) and, T_I and T_C are the periods associated with the assumed impulsive and convective motions of the structure and the fluid, respectively.

4.2.2 Overturning Moment

The overturning moment at the base of the tank is determined for the impulsive and the convective components as follows:

$$M_I = C_{SI}(W_W h_W + W_R h_R + W_I h_I) \qquad \text{Impulsive}$$

$$M_C = C_{SC}(W_C h_C) \qquad \text{Convective}$$

h_W = height at which inertia of wall is assumed to act

Total overturning moment $M_T = \sqrt{M_I^2 + M_C^2}$

The overturning moments due to impulsive ($W_I h_I$) and convective ($W_C h_C$) components should include the effect of base pressure where necessary. A method for including the base pressure is given in ACI 350.3[3-6].

4.2.3 Response Spectrum Method

The design response spectrum shown in Fig. 4-6 can be determined as follows:

$$S_a = S_{DS}\left[\frac{0.6T}{T_0} + 0.4\right] \text{ for } T < T_0$$

$$S_a = S_{DS} \qquad \text{for } T_0 < T < T_S$$

$$S_a = \frac{S_{D1}}{T} \qquad \text{for } T > T_S$$

where $T_0 = 0.2S_{D1}/S_{DS}$ and $T_S = S_{D1}/S_{DS}$

The above equations can be used to determine the response ordinate S_{aI} for impulsive motion using T_I and S_{aC} for convective motion using T_C. The damping corresponding to the convective motion is of the order of 0.5 to 2% as compared to 5% assumed for impulsive motion. The spectrum shown in Fig. 4-6 is for 5% damping. Note that ACI 350.3 recommends 0.5% damping when calculating the convective forces. This requires multiplying the design spectral acceleration coefficent S_{D1} by a factor of 1.5. Note that for large periods (T_C>2.4 sec) ACI 350.3 recommends modifying the design equations to better estimate the long-period excitation effect. Based on these recommendations, the following equations should be considered with IBC 2000:

$$\text{for } T_C < 4 \text{ sec, } S_{aC} = \frac{1.5S_{D1}}{T}$$

$$\text{for } T_C \ge 4 \text{ sec, } S_{aC} = \frac{6S_{D1}}{T_C^2}$$

When site specific response spectrum are used, the values of S_{DS} and S_{D1} shall not be less than 80% of the values obtained from the general procedure of Section 1615.1 (see Section 3.1.2 of this publication).

In certain situations, a site-specific response spectrum is required. Section 1615.2 gives provisions for using the site-specific response spectrum. This spectrum is to be based on the maximum considered earthquake ground motion having a 2% probability of being exceeded in 50 years.

The base shear is computed using values S_{aI} and S_{aC} from the spectrum as follows:

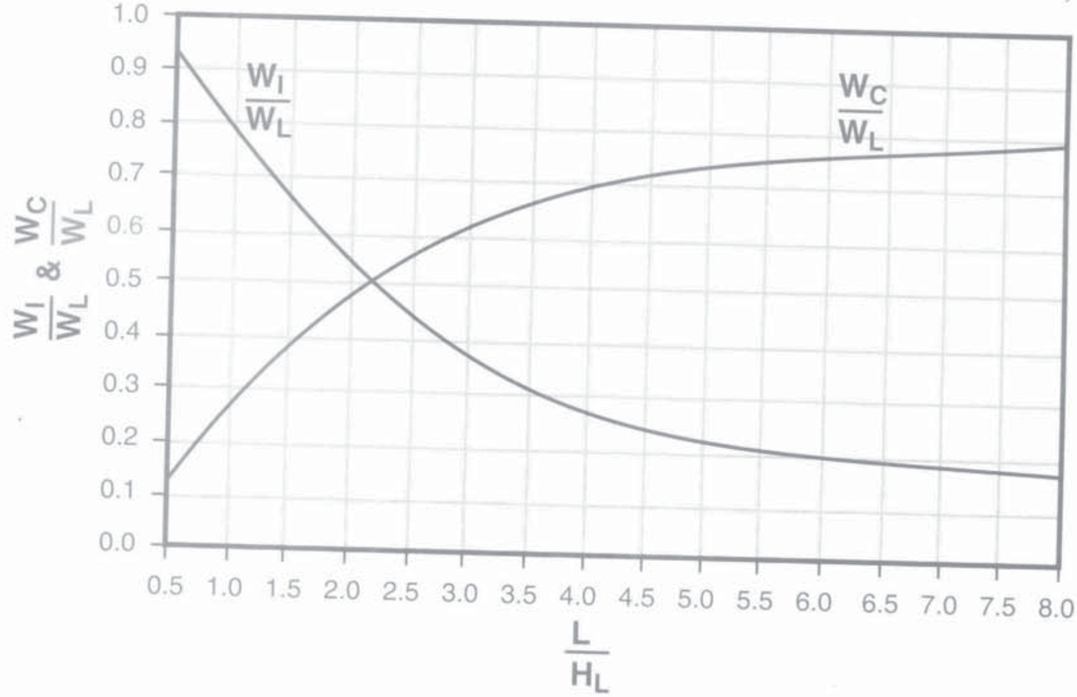
$$V_I = \frac{S_{aI}I}{R}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

$$V_C = \frac{S_{aC}I}{R}(W_C) \qquad \text{Convective}$$

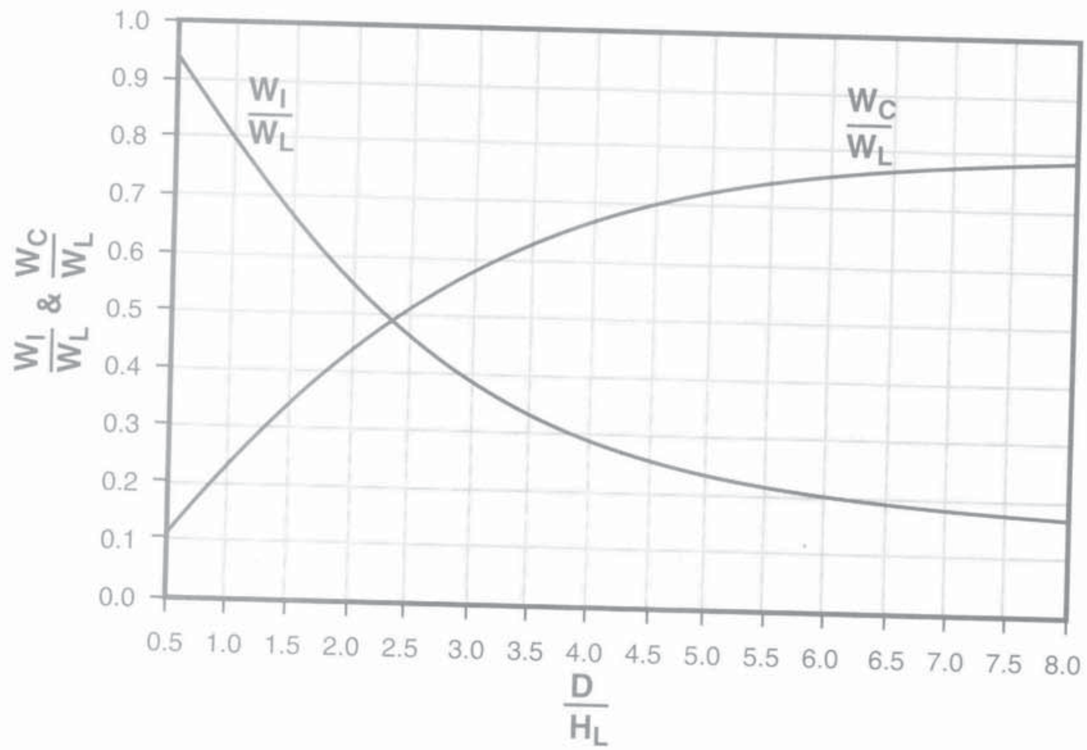
Total base shear $V_T = \sqrt{V_I^2 + V_C^2}$

$\ge 0.8\ V_T$

calculated using Section 4.2.1

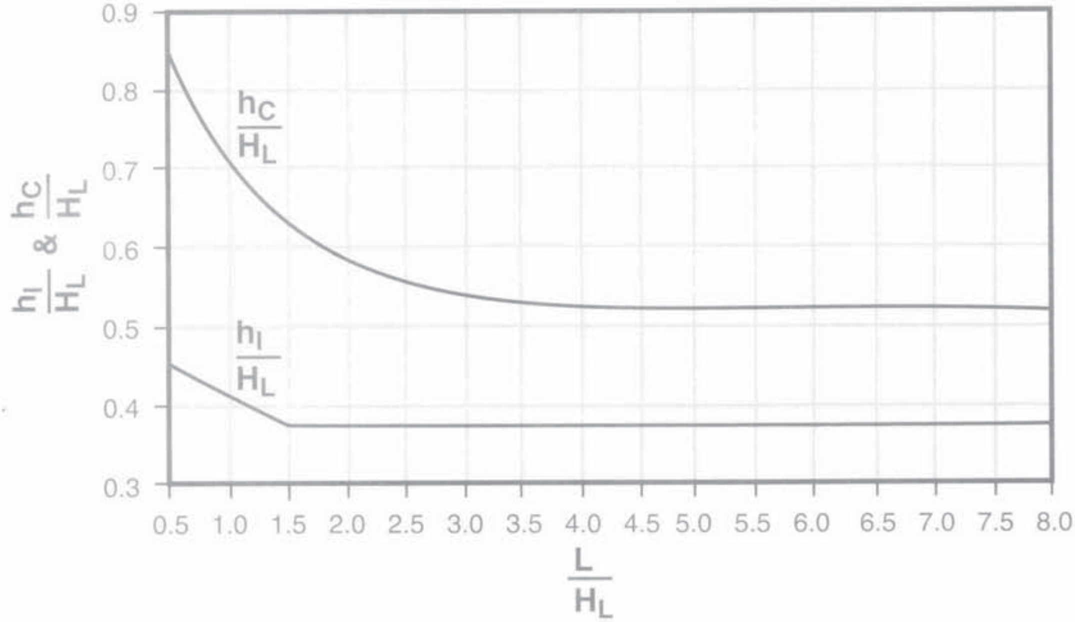


(a) Rectangular Tanks

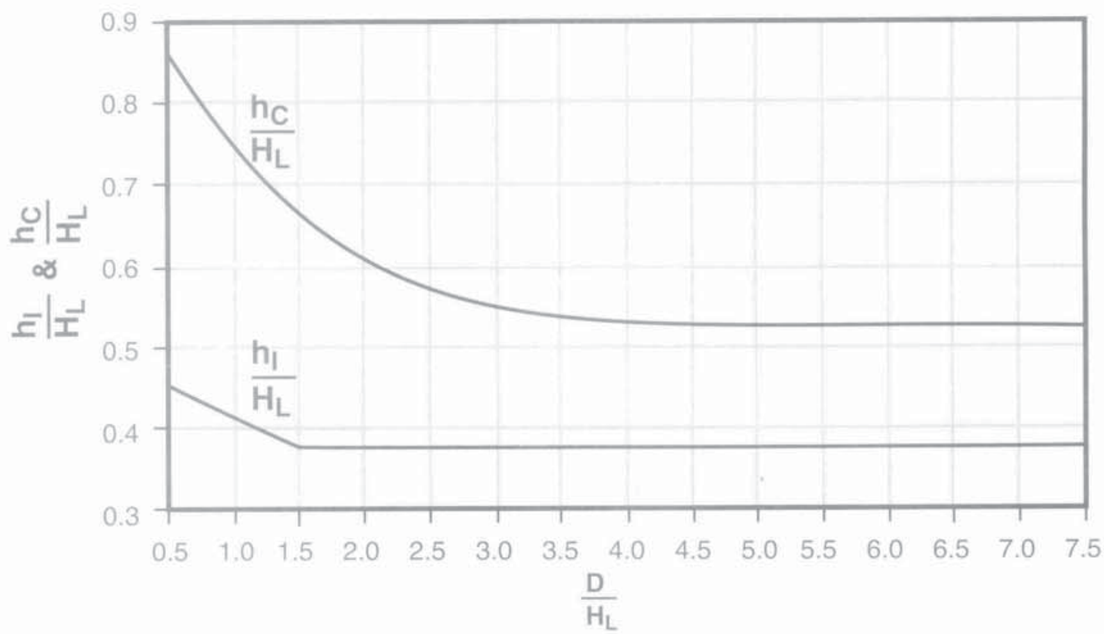


(b) Circular Tanks

Fig. 4-4 Impulsive and Convective Weights for (a) Rectangular and (b) Circular Tanks (Adapted from Ref. 3-6)



(a) Rectangular Tanks



(b) Circular Tanks

Fig. 4-5 Effective Height of Impulsive and Convective Weights for (a) Rectangular and (b) Circular Tanks (Adapted from Ref. 3-6)

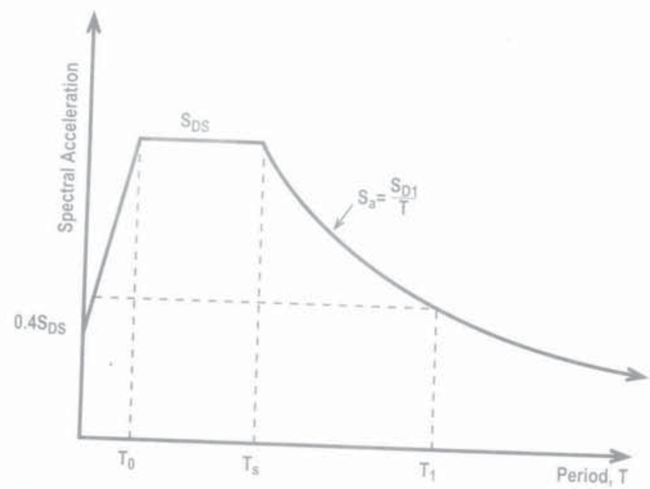


Fig. 4-6 Design Response Spectrum, IBC-2000

4.3 UBC '97 METHOD

4.3.1 Base Shear

Section 3.2 of this publication gives a procedure that assumes liquid-containing structures to be essentially rigid. It does not allow separate computations of impulsive and convective components of the base shear. To better estimate the design forces, the general procedure for base shear given for building structures (UBC 1630.2) is used in combination with the provisions of ACI 350.3[3-6], as follows:

$$V_I = \frac{C_V I}{RT_I}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

In the short period range, the impulsive base shear need not be greater than

$$V_I = \frac{2.5 C_a I}{R}(W_W + W_R + W_I)$$

In the long period range, the impulsive base shear shall not be less than

$$V_I = 0.11 C_a I \; (W_W + W_R + W_I)$$

Also, for structures located close to the potential sources of earthquakes in Seismic Zone 4, the impulsive base shear shall not be less than

$$V_I = \frac{0.8 Z N_V I}{R}(W_W + W_R + W_I)$$

$$V_C = \frac{C_V I}{RT_C}(W_C) \qquad \text{Convective}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2}$

The quantites C_V and C_a depend upon the Zone Factor Z and the soil profile type. The values for these parameters are given in Tables 16-Q and 16-R of the 1997 UBC, respectively and Z = seismic zone factor, which represents the maximum effective peak acceleration (EPA) corresponding to a site-specific ground motion having a 90% probability of not being exceeded in a 50-year period.

I = importance factor (Table 16-K, UBC '97).

R = response modification factor. UBC '97 gives some R values for liquid-containing structures in Table 16-P.

W_W, W_R, W_I and W_C represent the wall weight, roof weight, the impulsive weight and the convective weight respectively. The impulsive weight W_I and convective weight W_C can be determined as a fraction of the total liquid weight of rectangular or circular tank from Fig. 4-4a and 4-4b, respectively. The height at which the impulsive and convective weights are assumed to act can be determined from Figs. 4-5a and 4-5b for rectangular and circular tanks, respectively.

4.3.2 Overturning Moment

$$M_I = \frac{C_V I}{RT_I}(W_W h_W + W_R h_R + W_I h_I) \qquad \text{Impulsive}$$

$$\leq \frac{2.5 C_a I}{R}(W_W h_W + W_W h_R + W_I h_I)$$

$$M_C = \frac{C_V I}{RT_C}(W_C h_C) \qquad \text{Convective}$$

Total overturning moment $M_T = \sqrt{M_I^2 + M_C^2}$

h_W = height at which inertia of wall is assumed to act.

The overturning moments due to impulsive $(W_I h_I)$ and convective $(W_C h_C)$ components should include the effect of base pressure where necessary. A method for including the base pressure is given in ACI 350.3[3-6].

4.3.3 Response Spectrum Method

The design response spectrum shown in Fig. 4-7 can be determined as follows:

$$S_a = C_a\left[\frac{1.5T}{T_0} + 1\right] \qquad \text{for } T < T_0$$

$$S_a = 2.5\,C_a \qquad \text{for } T_0 < T < T_S$$

$$S_a = \frac{C_V}{T} \qquad \text{for } T > T_S$$

where

$$T_S = C_V / 2.5 C_a$$

$$T_0 = 0.2 T_S$$

The above spectrum can be used to determine the response ordinate S_{aI} for impulsive motion using T_I and S_{aC} for convective motion using T_C. The spectrum is derived for 5% damping. For convective response a method similar

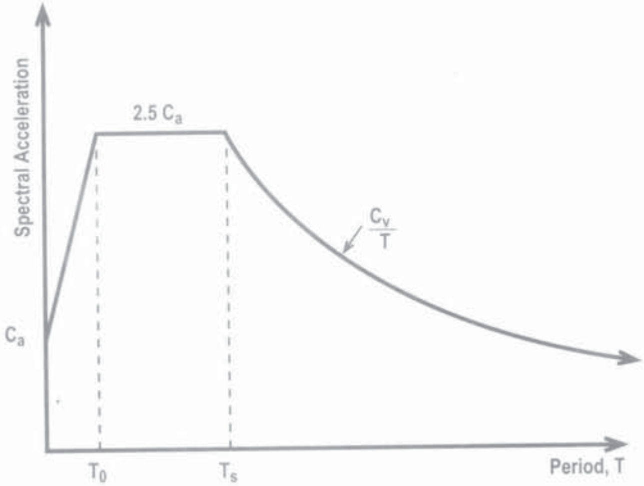


Fig. 4-7 Design Response Spectrum, UBC-1997

to that described in Section 4.2.3 may be considered for 0.5% damping and long-period excitation effect.

The site-specific response spectrum is constructed for ground motions that have a 10% maximum probability of exceedence in 50 years for 5% damping.

The impulsive and convective components of the base shear are determined using the spectral values S_{aI} and S_{aC} as follows:

$$V_I = \frac{S_{aI} I}{R}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

$$V_C = \frac{S_{aC} I}{R}(W_C) \qquad \text{Convective}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2}$
$\geq 0.8\ V_T$
obtained by using Section 4.3.1

4.4 UBC '94 METHOD

4.4.1 Base Shear

$$V_I = \frac{ZIC_I}{R_W}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

$$V_C = \frac{ZIC_C}{R_W}(W_C) \qquad \text{Convective}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2} \ \geq \ 0.075\ Z\ I\ W_I$

The impulsive weight W_I and convective weight W_C can be determined as a fraction of the total liquid weight of rectangular or circular tank from Fig. 4-4a and 4-4b, respectively. The height at which the impulsive and convective weights are assumed to act can be determined from Figs. 4-5a and 4-5b for rectangular and circular tanks, respectively.

The parameters C_I and C_C are determined as follows:

$$C_I = \frac{1.25S}{T_I^{2/3}} \qquad \text{Impulsive}$$

$$C_C = \frac{1.25S}{T_C^{2/3}} \qquad \text{Convective}$$

$0.075\ R_W \leq C_I \leq 2.75$

4.4.2 Overturning moment

$$M_I = \frac{ZIC_I}{R_W}(W_W h_W + W_R h_R + W_I h_I) \qquad \text{Impulsive}$$

$$M_C = \frac{ZIC_C}{R_W}(W_C h_C) \qquad \text{Convective}$$

Total overturning moment $M_T = \sqrt{M_I^2 + M_C^2}$

h_W = height at which inertia of wall is assumed to act.

The uplift pressure at the base can increase the overturning moments and effect the stability of the tank.

The overturning moments due to impulsive ($W_I h_I$) and convective ($W_C h_C$) components should include the effect of base pressure where necessary. A method for including the base pressure is given in ACI 350.3[3-6].

4.4.3 Response Spectrum Method

The design force on the tank can also be determined from a design spectrum such as the one given in UBC[3-2], or by using the site-specific response spectrum. The site-specific response spectrum is constructed for ground motions that have a 10% maximum probability of exceedence in 50 years for 5% damping. The design base shear is determined using the spectrum as follows:

$$V_I = \frac{S_{aI} I}{R_W}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

$$V_C = \frac{S_{aC} I}{R_W}(W_C) \qquad \text{Convective}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2}$

The quantities S_{aI} and S_{aC} are the impulsive and convective spectral acceleration coefficients which correspond to the impulsive and convective periods T_I and T_C from the simplified spectrum (Fig. 4-8). The design spectrum is given in UBC Fig. 16-3. The base shear computed using the site-specific spectrum is not permitted to be less than 90% of the value determined using the static force procedure given under Section 4.4.1.

For convective response a method described in ACI 350.3 may be considered to better estimate the effect of 0.5% damping and long-period excitation effect.

4.5 BOCA AND SBC METHOD

4.5.1 Base Shear

$$V_I = C_{SI}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

$$V_C = C_{SC}(W_C) \qquad \text{Convective}$$

$$C_{SI} = \frac{1.2A_V S}{RT_I^{2/3}} \le \frac{2.5A_a}{R} \qquad \text{Impulsive}$$

$$C_{SC} = \frac{1.2A_V S}{RT_C^{2/3}} \le \frac{2.5A_a}{R} \qquad \text{Convective}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2}$

where A_a and A_v are the effective peak acceleration coefficient and the effective peak velocity-related acceleration coefficient, respectively (see Section 1610.1.3 of BOCA and Section 1607.1.5 of SBC)

The values of site coefficient (S) and response modification factor (R) should be taken from appropriate edition of the BOCA or SBC codes.

4.5.2 Overturning moment

$$M_I = C_{SI}(W_W h_W + W_R h_R + W_I h_I) \qquad \text{Impulsive}$$

$$M_C = C_{SC}(W_C h_C) \qquad \text{Convective}$$

Total overturning moment $M_T = \sqrt{M_I^2 + M_C^2}$

4.6 ACI 350.3-01 METHOD

The provisions of ACI 350.3[3-6] are to be used in conjunction with Chapter 21 (Special Provisions for Seismic Design) of ACI 350-01. These provisions are compatible with UBC 1994. Note that ACI 350-01 is based on ACI 318-95[4-3] for most of its design provisions and load combinations. Section 21.2.1.7 of ACI 350-01 indicates that the environmental durability factor (S) defined in Section 9.2.8 need not be applied to load combinations that include earthquake effects. The load combinations applicable under various Codes are given in Chapter 3 of this publication.

Where ACI 350-01 is adopted for use, the provisions of Chapter 21 along with ACI 350.3 and ACI 318-95 are applicable. Note that ACI 350.3 also gives recommendations for seismic zone factors (Z), and soil factors (S), which are mostly consistent with UBC 1994. The importance factors of ACI 350.3-01 are given in Table 4-1. ACI 350.3-01 also gives separate response modification factors R_{WI} and R_{WC} for impulsive and convective motions of the liquid-containing structure (Table 4-2.)

4.7 PERIOD

The equations for determining the impulsive period T_I and convective period T_C of rectangular and circular liquid-

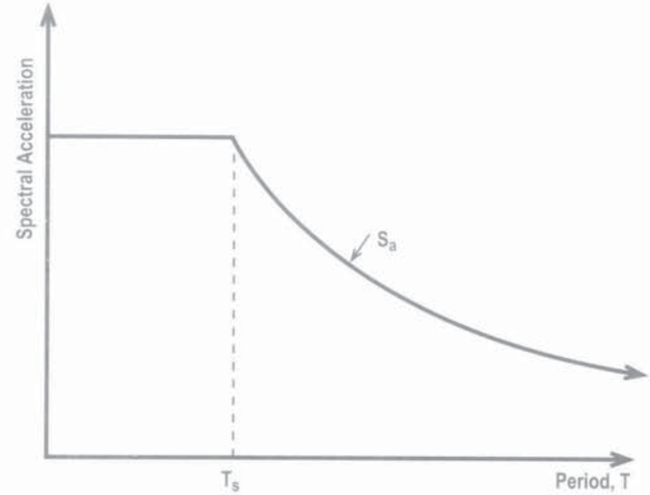


Fig. 4-8 Simplified Response Spectrum, UBC-1994

containing structures having different base conditions are given below. However, it is permitted to use any other rational method that includes a reasonable distribution of mass and stiffness characteristics for determining the natural period of the structure.

As most concrete tanks are relatively rigid, T_I may be taken as 0.3 seconds or less for the preliminary and approximate design calculations. It is recommended that for flexible base tanks, T_I should not exceed 1 second for anchored and unanchored contained tanks. This limit should not exceed 2 seconds for unanchored uncontained tanks. The limits on the periods suggested herein are to prevent excessive deformation of tanks.

4.7.1 Rectangular Tanks

The following equation can be used to determine the impulsive period of a rectangular tank:

$$T_I = 2\pi\sqrt{\frac{W}{gK}}$$

For fixed-base constant thickness cantilever walls:

$$K = \frac{E_C}{48}\left(\frac{t_w}{h}\right)^3$$

$$W = W_W + W_R + W_I \quad \text{(kips)}$$

where h = mean height (ft) at which the inertia force of the tank and its contents is assumed to act, t_w = wall thickness (in.), E_c = modulus elasticity of concrete (ksi) g = acceleration due to gravity (ft/sec^2) and K = stiffness coefficient (kips/ft).

The period associated with the convective component (T_C) can be determined as follows:

$$T_C = \frac{2\pi}{\lambda}\sqrt{L}$$

Table 4-1 Importance Factor, *I* (Table 4(c), Ref. 3-6)

Tank Use	Factor *I*
Tanks containing hazardous materials [(1)]	1.5
Tanks that are intended to remain usable, for emergency purposes after an earthquake; or tanks that are part of lifeline systems	1.25
All other tanks	1.0

(1) For tanks containing hazardous materials, engineering judgment may require a factor I > 1.5 to account for the possibility of an earthquake greater than the design earthquake.

Table 4-2 Response Modification Factor, R_w (Table 4(d), Ref. 3-6)

Type of Structure	R_{wi}		R_{wc}
	On or Above Grade	Buried[(1)]	
(a) Anchored, flexible-base tanks	4.5	4.5[(2)]	1.0
(b) Fixed or hinged-base tanks	2.75	4.0	1.0
(c) Unanchored, contained or uncontained tanks[(3)]	2.0	2.75	1.0
(d) Elevated Tanks	3.0	—	1.0

(1) *Buried tank is defined as a tank whose maximum water surface is at or below ground level.*
(2) *R_{wi} = 4.5 is the maximum R_{wi} value permitted to be used for any liquid-containing concrete structure.*
(3) *Unanchored, uncontained tanks shall not be built in Zone 2B or higher.*

where $\frac{2\pi}{\lambda}$ can be obtained from Fig. 4-9(a) for a given L/H_L of the tank. (L = length of tank in direction of analysis (ft)).

4.7.2 Circular Tanks

(a) **Non-sliding Base**. The following equations can be used to determine the impulsive period of fixed or hinged base circular tanks with or without prestressing:

$$T_I = \frac{2\pi}{\omega_I}$$

where

$$\omega_I = C_L \frac{12}{H_L}\sqrt{\frac{E_C}{\rho_C}}$$

$$C_I = 10C_W\sqrt{\frac{t_w}{12r}}$$

and ρ_c = mass density of concrete (4.66 lb-sec²/ft⁴), t_w = thickness of wall (in.), r = radius of tank (ft), E_C = modulus of elasticity of concrete (lb/in.²), C_W is given in Fig. 4-10 in terms of D/H_L.

(b) **Flexible Base**. The following equations can be used to determine the impulsive period T_I of flexible base circular prestressed tanks:

$$T_I = \sqrt{\frac{8\pi W}{gDk_a}} \quad \leq \text{ 1 second for anchored tanks and 2 seconds for unanchored tanks}$$

Note that ACI 350.3 specifies a limit of 1.25 seconds on both anchored and unanchored maximum periods.

$$W = W_W + W_R + W_I$$

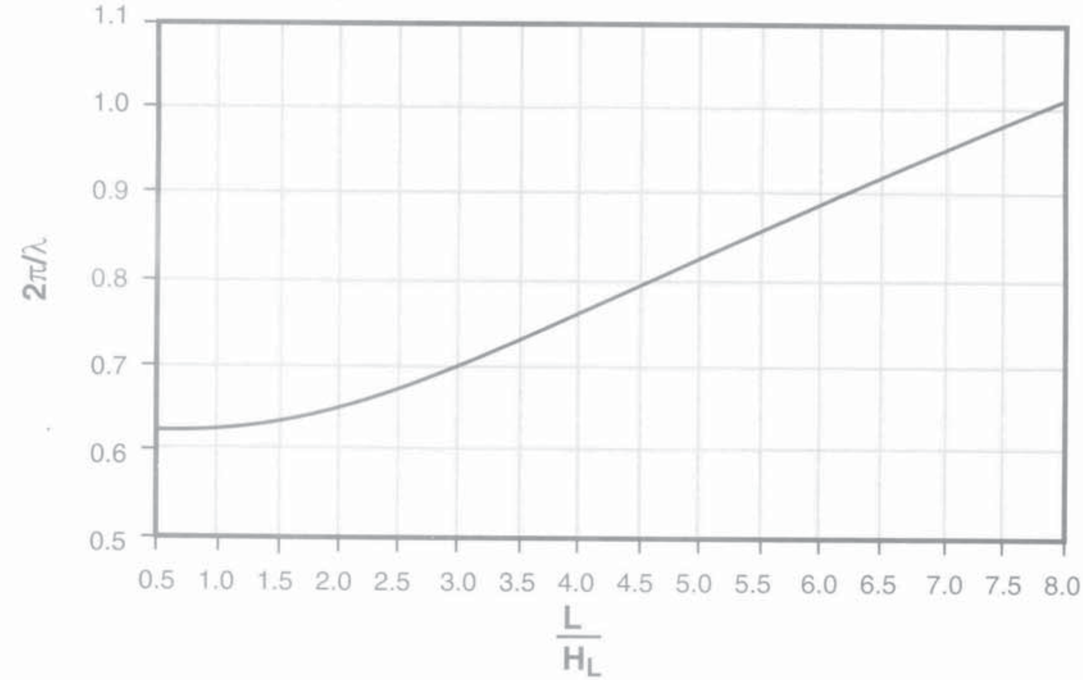
$$k_a = 144\left[\frac{A_s E_s \cos^2\beta}{L_s S_b} + \frac{2G_p w_p L_p}{t_p S_p}\right] \quad \text{For anchored flexible tanks}$$

$$k_a = 144\left[\frac{2G_p w_p L_p}{t_p S_p}\right] \quad \text{For unanchored flexible tanks}$$

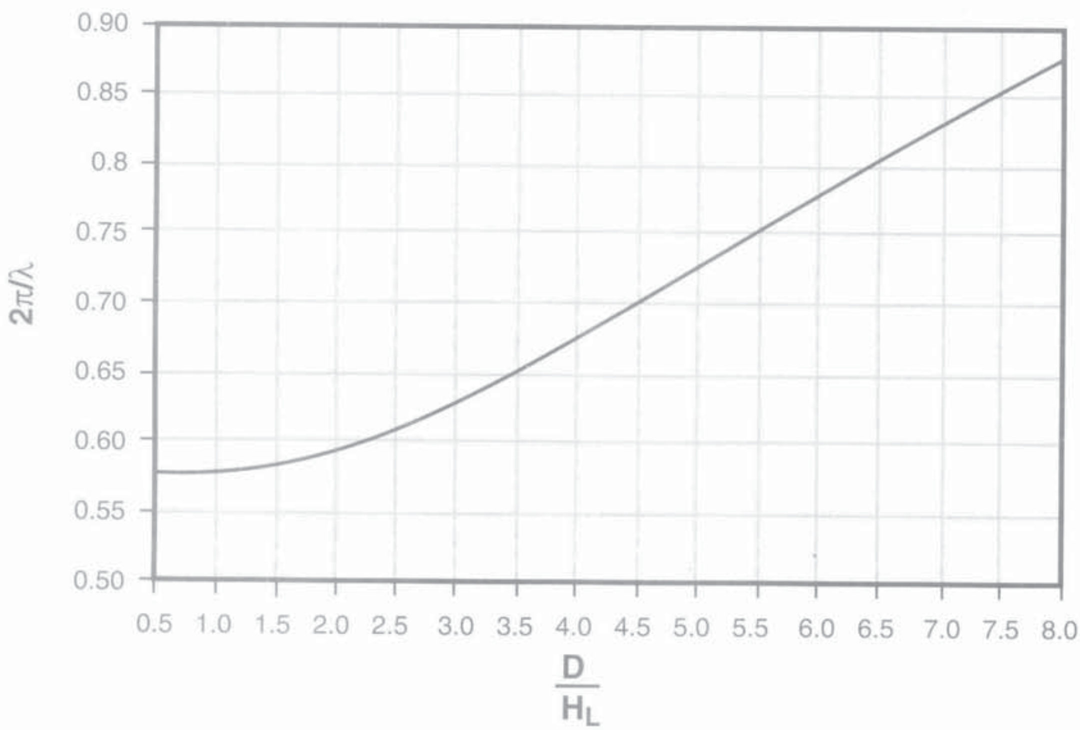
where A_s = cross-sectional area of cable/strand (in.²), E_s = modulus of elasticity of cable/strand (ksi), β = angle of cable/strand with horizontal, L_s = effective length of cable/strand taken as sleeve length plus 35 times the diameter (in.), S_b = spacing between cable sets (in), S_p = spacing of elastomeric pads (in), G_p = shear modulus of elastomeric pads (ksi), t_p = thickness of elastomeric bearing pad (in.), L_p = length of individual elastomeric pad (in.) and w_p = width of elastomeric pad in radial direction (in.), and k_a = spring constant (k/ft²).

The convective period T_C for both non-sliding and flexible base tanks can be determined using the following equation:

$$T_C = \frac{2\pi}{\lambda}\sqrt{D}$$



(a) Rectangular Tanks



(b) Circular Tanks

Fig. 4-9 Charts for Obtaining Factor $\frac{2\pi}{\lambda}$ for Computation of Convective Period (T_c) for (a) Rectangular and (b) Circular Tanks (Adapted from Ref. 3-6)

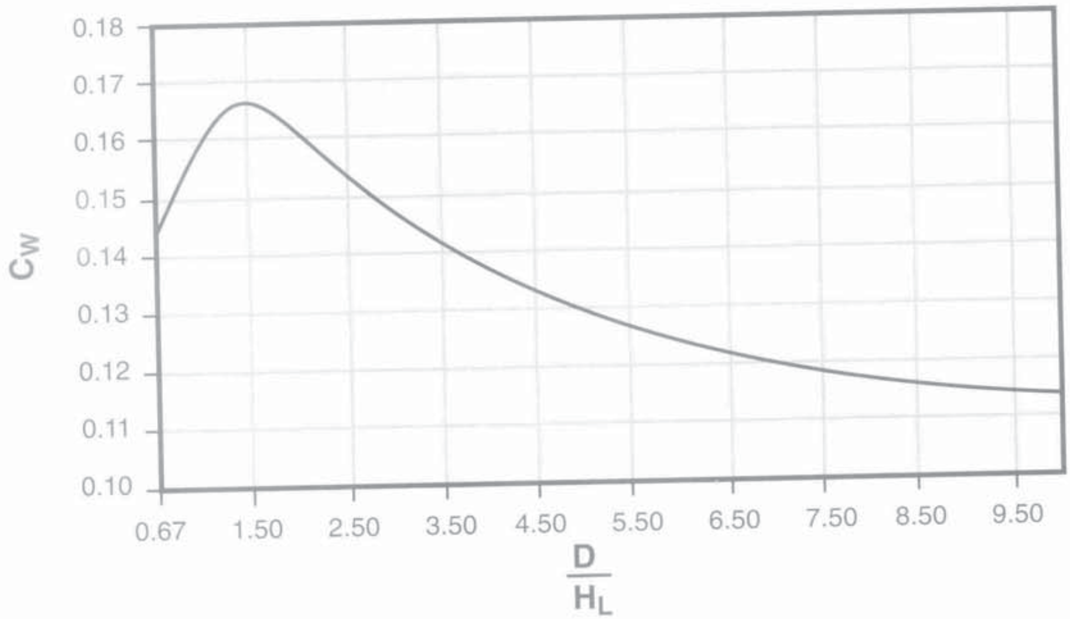


Fig. 4-10 Chart for Obtaining Factor C_w for Computation of Impulsive Period (T_i) of a Non-Sliding Circular Tank (Adapted from Ref. 3-6)

The value of $\frac{2\pi}{\lambda}$ may be obtained from Fig. 4-9(b) for D/H_L of a tank for both rigid and flexible base tanks (D = diameter of tank (ft), H_L = height of liquid (ft)).

4.8 VERTICAL ACCELERATION

The effect of vertical accelerations should be included in the design of tank components. In the absence of more detailed analysis, the magnitude of vertical acceleration is generally taken as two-thirds of the horizontal acceleration. The effects of vertical acceleration as recommended in ACI 350.3 are computed as follows:

The hydrodynamic pressure per foot height of the tank is

$$p_{hy} = \ddot{u}_V q_{hy}$$

where $q_{hy} = \gamma_L(H_L\text{-}y)$ lbs/ft, unit hydrostatic pressure at level y above tank base

$\ddot{u}_v$ = magnitude of vertical acceleration associated with the vertical period (T_V) of the structure and γ_L = specific weight of contained fluid.

The period associated with the vertical motion (T_V) of the circular tank is computed as follows:

$$T_V = 2\pi\sqrt{\frac{\gamma_L D H_L^2}{24 g t_W E_C}}$$

4.9 FREEBOARD

The anticipated unrestrained sloshing height should be computed to determine any sloshing pressure on the tank roof, wall and the joint between roof and the wall. Note that tanks with inadequate freeboard will experience uplift pressures on the roof due to liquid sloshing. Tanks in seismic zones 3 and 4 and tanks designed for importance factor greater than 1.0 should either have adequate freeboard d_{max} (Fig. 4-2a) or should be designed for the forces due to restrained sloshing and vertical acceleration effects. The sloshing height may be computed by using the following equations which are based on concepts similar to those given in ACI 350.3.

IBC 2000 Method

$$d_{max} = \frac{S_{D1}I}{1.4T_C}\left(\frac{L}{2}\right) \quad \text{For Rectangular Tanks}$$

$$d_{max} = \frac{S_{D1}I}{1.4T_C}\left(\frac{D}{2}\right) \quad \text{For Circular Tanks}$$

UBC '97 Method

$$d_{max} = \frac{C_V I}{1.4T_C}\left(\frac{L}{2}\right) \quad \text{For Rectangular Tanks}$$

$$d_{max} = \frac{C_V I}{1.4T_C}\left(\frac{D}{2}\right) \quad \text{For Circular Tanks}$$

UBC '94 Method

$$d_{max} = ZC_C I\left(\frac{L}{2}\right) \quad \text{For Rectangular Tanks}$$

$$d_{max} = ZC_C I\left(\frac{D}{2}\right) \quad \text{For Circular Tanks}$$

BOCA and SBC Method

$$d_{max} = \frac{1.2A_V S}{1.4T_C^{\ 2/3}}\left(\frac{L}{2}\right) \quad \text{For Rectangular Tanks}$$

$$d_{max} = \frac{1.2A_V S}{1.4T_C^{\ 2/3}}\left(\frac{D}{2}\right) \quad \text{For Circular Tanks}$$

4.10 EARTH PRESSURE

Effect of earth pressure should be included both in the base shear computation and for design of walls of a partially or fully buried liquid-containing structure. The effect of ground water, if any, should also be taken into consideration. Active earth pressure is caused as a result of the structure moving away from the surrounding soil while passive pressure results due to the structure moving into the surrounding soil (Fig. 4-11). Table 4-3 gives the approximate magnitude of movement required to reach the minimum active and maximum passive pressure condition[4-4, 4-5].

ACI 350.3 stipulates that in computing the earth pressure, the coefficient of dynamic lateral earth pressure k_o at rest should be used unless it is determined that the structure deflects sufficiently to warrant use of active and passive pressure k_a and k_p, respectively. The coefficient k_o varies from 0.4 - 0.6 for cohesionless soils and 0.4 - 0.8 for cohesive soils[4-6]. The resultant of the seismic component of the earth pressure can be assumed to act at a height 0.6 times the earth height above base.

Note that the above simplification of the earth pressure effects is based on the assumption that the liquid-containing structure will not deflect enough to result in active or passive pressure due to the seismic excitation. When this is not the case, dynamic active and passive pressures need to be calculated. References 4-4 and 4-5 give the guidelines for computing the dynamic active and passive pressures. In situations where detailed active and passive pressure computations are deemed necessary, the user should also refer to Okabe (Ref. 4-7) and Mononobe and Matsuo (Ref. 4-8).

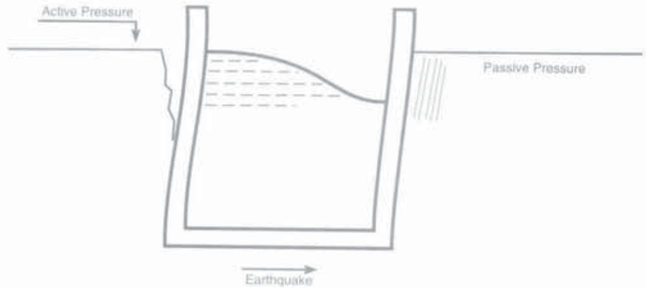


Fig. 4-11 Active and Passive Earth Pressure due to Seismic Movement of Tank

Table 4-3. Approximate Magnitude of Movement Required to Reach Minimum Active and Maximum Passive Earth Pressure (Ref. 4-4)

Type of Backfill	Δ/H^*	
	Active	Passive
Dense Sand	0.001	0.01
Medium-dense Sand	0.002	0.02
Loose Sand	0.004	0.04

*Δ = movement at top of wall and H = height of wall.

4.11 REFERENCES

4-1. Housner, G. W., "The Dynamic Behavior of Water Tanks," *Bulletin of the Seismological Society of America*, Vol. 53, No. 2, 1963, pp. 381-387.

4-2. Haroun, M. A. and Housner, G. W., "Seismic Design of Liquid Storage Tanks," *Journal of the Technical Councils of the ASCE, Procedings of the American Society of Civil Engineers, ASCE*, Vol. 107, No. TCI, 1994, pp. 191-207.

4-3. *Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)*, American Concrete Institute, Farmington Hills, MI, 1995.

4-4. Clough, G. W., and Duncan, J. M., (1991), Chapter 6: Earth Pressures, in Foundation Engineering Handbook, Second Edition, NY, pp. 223-235.

4-5. Ebeling, R. M. and Morrison, E. E., "The Seismic Design of Waterfront Structures," *NCEL Technical Report, TR-939*, Naval Civil Engineering Laboratory, Port Hueneme, CA, 1993.

4-6. Bowles, J. E., *Foundation Analysis and Design*, 4th Ed., McGraw-Hill, Inc., NY, 1988.

4-7. Okabe, S. (1926), "General Theory of Earth Pressures," *Journal Japan Society of Civil Engineering*, Vol. 12, No. 1.

4-8. Mononobe, N., and Matsuo, H., (1929), "On the Determination of Earth Pressures During Earthquakes," *Proceedings World Engineering Congress*, 9.

CHAPTER 5

Design of Components

5.1 GENERAL

All components of liquid-containing structures such as walls, roof slab, base slab, joints, baffle walls and piping fixtures must be designed for the maximum effects of stresses produced by different applicable loads. This will ensure the intended overall performance of the liquid-containing structure. The design of some of the components is not straightforward due to the complexity of the stress distribution, particularly in the case of circular tanks. In the absence of a more refined analysis, approximate methods given in this chapter may be used to design these components.

Figure 5-1 shows the hydrodynamic forces on walls and base slab of a tank due to earthquake ground motion. The hydrodynamic forces include the effect of impulsive and convective motions of the contained liquid. Besides hydrodynamic forces, the tank elements are to be designed for their own inertia forces and the forces transferred from other elements.

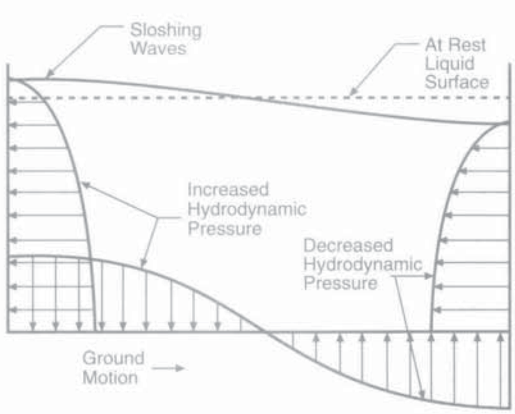


Fig. 5-1. Hydrodynamic Pressures Due to Ground Motion.

5.2 DESIGN OF TANK WALLS

5.2.1 Wall Forces

The walls should be designed for the combined effects of static and dynamic loads per the applicable load combinations given in Chapter 3. The static loads include the dead load, live load, hydrostatic pressure and earth pressure. The dynamic loads include the inertia of the elements, the hydrodynamic forces (impulsive and convective components of fluid motion) and dynamic earth pressure.

The inertia, impulsive and convective forces on the walls of rectangular and circular tanks can be determined for different codes as follows:

IBC 2000 Method

$$P_W = \frac{S_{DS}I}{R}W_W \le \frac{S_{D1}I}{RT_I}W_W \qquad \text{Wall Inertia}$$

$$P_R = \frac{S_{DS}I}{R}W_R \le \frac{S_{D1}I}{RT_I}W_R \qquad \text{Roof Inertia}$$

$$P_I = \frac{S_{DS}I}{R}W_I \le \frac{S_{D1}I}{RT_I}W_I \qquad \text{Impulsive}$$

$$P_C = \frac{S_{DS}I}{R}W_C \le \frac{S_{D1}I}{RT_C}W_C \qquad \text{Convective}$$

UBC 1997 Method

$$P_W = \frac{C_V I}{RT_I}W_W \qquad \text{Wall Inertia}$$

$$P_R = \frac{C_V I}{RT_I}W_R \qquad \text{Roof Inertia}$$

$$P_I = \frac{C_V I}{RT_I}W_I \qquad \text{Impulsive}$$

$$P_C = \frac{C_V I}{RT_C}W_C \qquad \text{Convective}$$

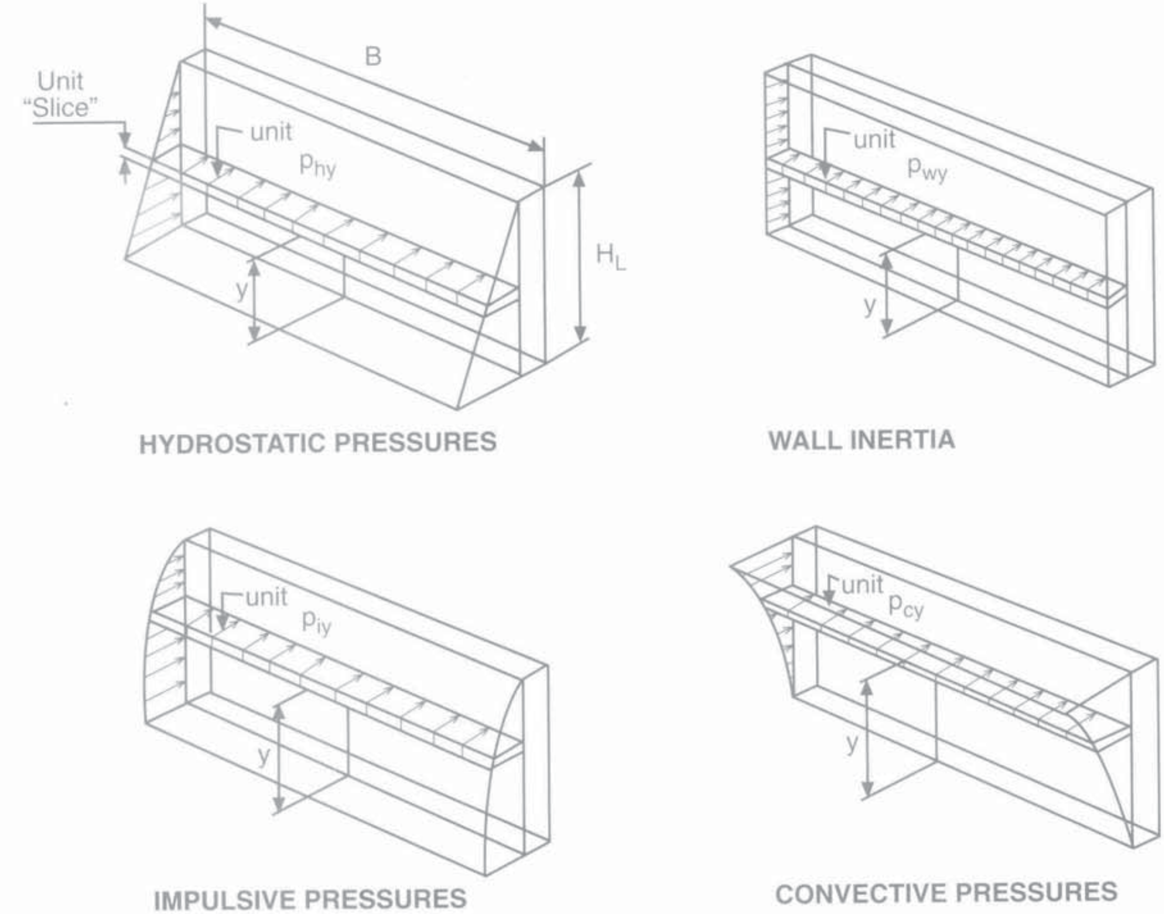


Fig. 5-2. Distribution of Hydrostatic, Inertia, Impulsive, and Convective Pressure on Wall of a Rectangular Liquid–Containing Structure (Adapted from Ref. 3-6)

where

$$0.11C_aI \le \frac{C_VI}{RT_I} \le \frac{2.5C_aI}{R}$$

$$\frac{0.8ZN_VI}{R} \le \frac{C_VI}{RT_I} \qquad \text{Zone 4}$$

UBC 1994 Method

$$P_W = ZIC_I\frac{W_W}{R_W} \qquad \text{Wall Inertia}$$

$$P_R = ZIC_I\frac{W_R}{R_W} \qquad \text{Roof Inertia}$$

$$P_I = ZIC_I\frac{W_I}{R_W} \qquad \text{Impulsive}$$

$$P_C = ZIC_C\frac{W_C}{R_W} \qquad \text{Convective}$$

where

$$C_I = \frac{1.25S}{T_I^{2/3}} \le 2.75 \qquad \text{Impulsive}$$

$$C_C = \frac{1.25S}{T_C^{2/3}} \qquad \text{Convective}$$

BOCA and SBC Method

$$P_W = C_{SI}W_W \qquad \text{Wall Inertia}$$

$$P_R = C_{SI}W_R \qquad \text{Roof Inertia}$$

$$P_I = C_{SI}W_I \qquad \text{Impulsive}$$

$$P_C = C_{SC}W_C \qquad \text{Convective}$$

where

$$C_{SI} = \frac{1.2A_VS}{RT_I^{2/3}} \le \frac{2.5A_a}{R} \qquad \text{Impulsive}$$

$$C_{SC} = \frac{1.2A_VS}{RT_C^{2/3}} \le \frac{2.5A_a}{R} \qquad \text{Convective}$$

Fig. 5-3. Leading and Trailing Half of a Rectangular Tank

The forces at any height y due to inertia, impulsive and convective motions are given as follows:

$$P_{Wy} = \frac{P_W}{2H_W} \quad (P_W \text{ is based on weight of two walls})$$

$$P_{Iy} = P_I \frac{(4H_L - 6h_I) - (6H_L - 12h_I) \times y/H_L}{2H_L^2} \quad \text{Impulsive}$$

$$P_{Cy} = P_C \frac{(4H_L - 6h_C) - (6H_L - 12h_C) \times y/H_L}{2H_L^2} \quad \text{Convective}$$

5.2.2 Rectangular Tanks

The distribution of hydrostatic, inertia, impulsive and convective pressures on a rectangular wall are given in Fig. 5-2. For the purpose of design, the tank is divided into leading half and the trailing half portions as shown in Fig. 5-3. It is assumed that the impulsive and convective forces are equally resisted by the leading and the trailing walls perpendicular to the direction of the earthquake force. Thus, half of the total impulsive and convective force is assigned to each wall.

The leading and the trailing walls perpendicular to the earthquake force are designed for the combined effects of (1) wall inertia force, $\frac{P_W}{2}$ (2) one-half the impulsive force, P_I (3) one-half the convective force, P_C and (4) dynamic earth pressure, P_E against the buried portions of the tank, as shown in Fig. 5-4. Since earthquake forces are reversible, both the leading and the trailing walls should be designed for the maximum effects of these forces.

Section 5.2.1 outlines the procedure for determining the inertia (P_W), impulsive (P_I) and convective (P_C) forces on the wall depending upon the applicable building code. The dynamic earth pressure can be determined using Reference 4-5.

Note that convective force is out-of-phase with impulsive force because of the relatively large oscillation period of the contained liquid with respect to the tank motion. Therefore, square root of the sum of squares (SRSS) method should be used to combine the impulsive and convective forces. The dynamic earth pressure caused by the movement of the tank can be directly added to the impulsive effects for design purposes.

The vertical and horizontal bending and shear stresses in the walls may be determined using plate analysis given in Ref. 5-1.

Walls parallel to the earthquake force are designed for in-plane forces due to (a) their own inertia and, (b) reactions from the roof and abutting walls.

5.2.3 Circular Tanks

In case of circular tanks, the earthquake base shear is transmitted partially by membrane (tangential) shear and partially by radial shear that causes vertical bending. Actual distribution of stress can only be calculated through a finite element analysis. ACI 350.3 indicates that 80% of the base shear can be assumed to be transferred through tangential shear for tanks with D/H of 4. The remaining 20% will be transferred through vertical bending. The maximum tangential shear occurs at a point on tank wall

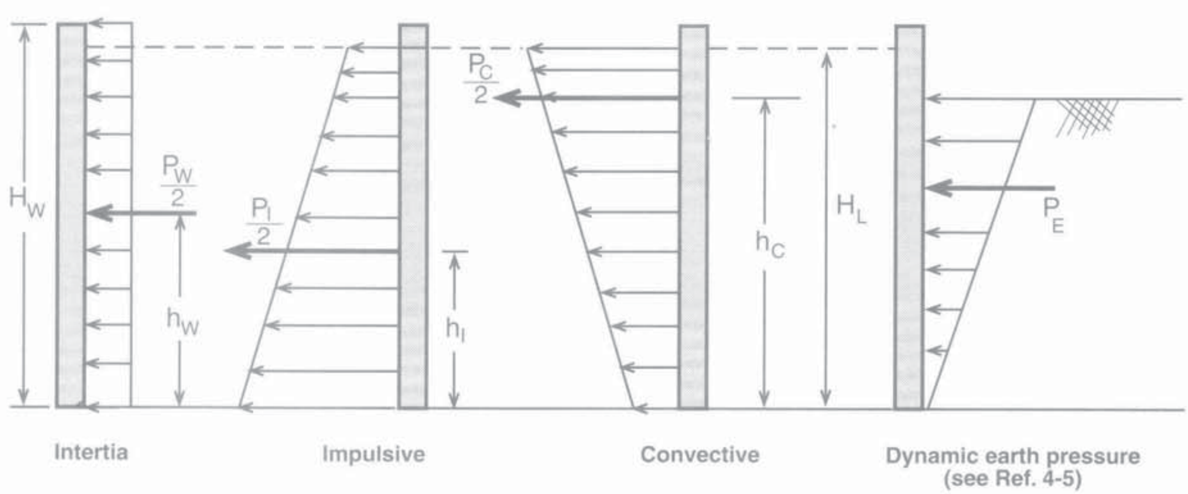


Fig. 5-4. Design Forces on Walls Perpendicular to the Earthquake Force

oriented 90 degrees from the direction of earthquake, as shown in Fig. 5-5.

For determining the forces/stresses in circular tank walls, the tank is divided into leading half and the trailing half portions as shown in Fig. 5-6. As far as dynamic loads are concerned, the cylindrical walls (Fig. 5-7) are designed for (a) wall inertia distributed uniformly around the entire circumference, (b) one-half the impulsive force P_I, applied symmetrically about an angle $\theta = 0$ and acting inward on one-half of the wall and one-half P_I applied symmetrical about $\theta = \pi$ and acting outward on the opposite half of the wall (c) one-half the convective force P_C applied symmetrically about an angle $\theta = 0$ and acting inward on one-half of the wall and one-half P_C applied symmetrical about $\theta = \pi$ and acting outward on the opposite half of the wall, and (d) the dynamic earth pressure, P_E against the buried portion of the tank.

Section 5.2.1 outlines the procedure for determining the inertia (P_W), impulsive (P_I) and convective (P_C) forces on the wall depending upon the applicable code. The forces P_{Wy}, P_{Iy} and P_{Cy}, are determined at height y above the base of the wall (see Section 5.2.1).

The horizontal distribution of the dynamic pressures at height y across the tank diameter D may be determined as follows:

$$p_{wy} = \frac{P_{Wy}}{\pi r}$$

$$p_{iy} = \frac{2P_{Iy}}{\pi r} cos\theta$$

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

The vertical bending and shear stresses in the walls can be computed using shell analysis (see Ref. 5-2). The hoop forces in cylindrical walls at any level y from the base can be determined by SRSS combination of the inertia, impulsive and convective stresses, as follows:

$$N_y = \sqrt{(N_{Wy} + N_{Iy})^2 + N_{Cy}^2}$$

Hoop stress: $\sigma_y = \dfrac{N_y}{12t_W}$

For circular tanks with flexible base, maximum forces N_{Wy}, N_{Iy}, and N_{Cy} can be determined for angle $\theta = 0$ (see Fig. 5-5), as follows:

$$N_{Wy} = P_{Wy}/\pi$$

$$N_{Iy} = 2P_{Iy}/\pi$$

$$N_{Cy} = 16P_{Cy}/9\pi$$

For non-flexible base tanks, the above equations should be modified to account for the effects of restraints.

5.3 DESIGN OF IMMERSED ELEMENTS

The immersed elements such as baffles, clarifier center wells, aerators, piping and launders must be designed for the effects of hydrodynamic forces. The immersed elements are subjected to additional forces due to the fact that the liquid surrounding them responds with them increasing their effective weight and the corresponding inertia force. The weight of liquid per lineal foot of height of the

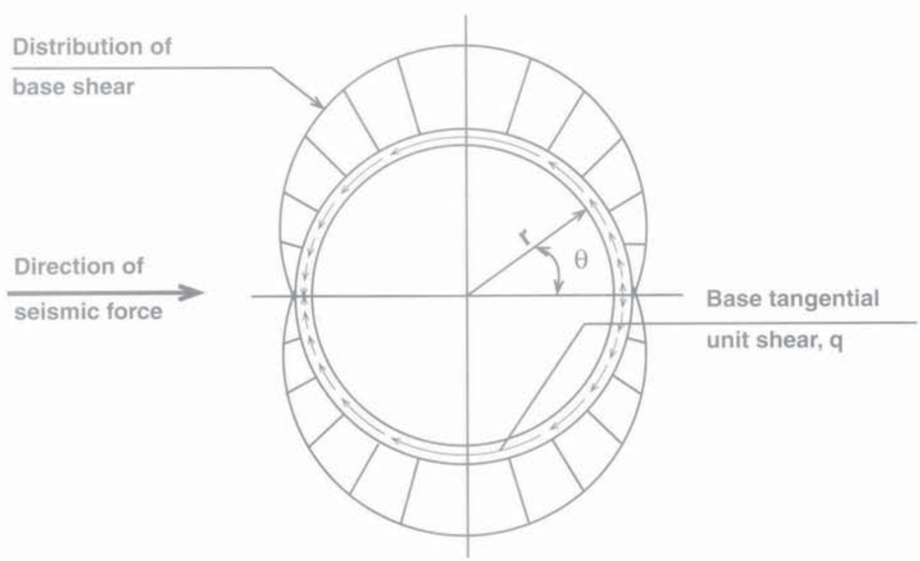


Fig. 5-5. Shear Transfer at Base (Adapted from Ref. 3-6)

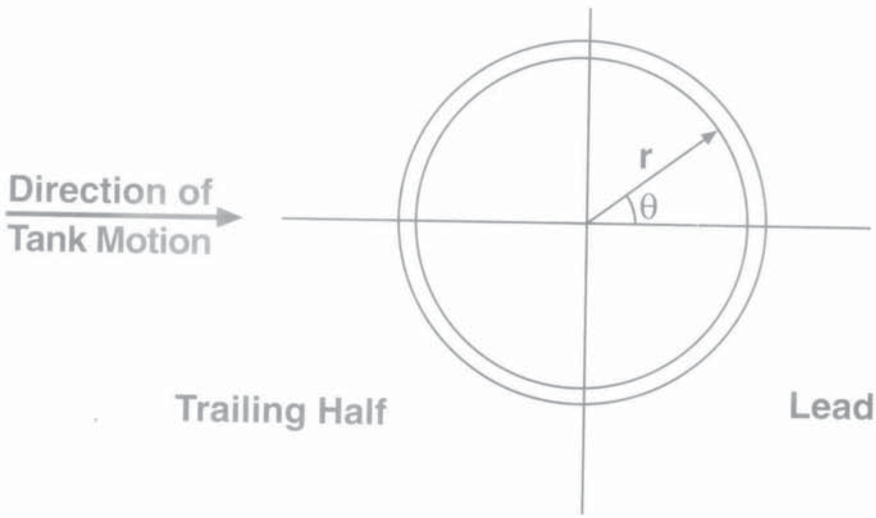


Fig. 5-6. Leading and Trailing Half of Circular Tank

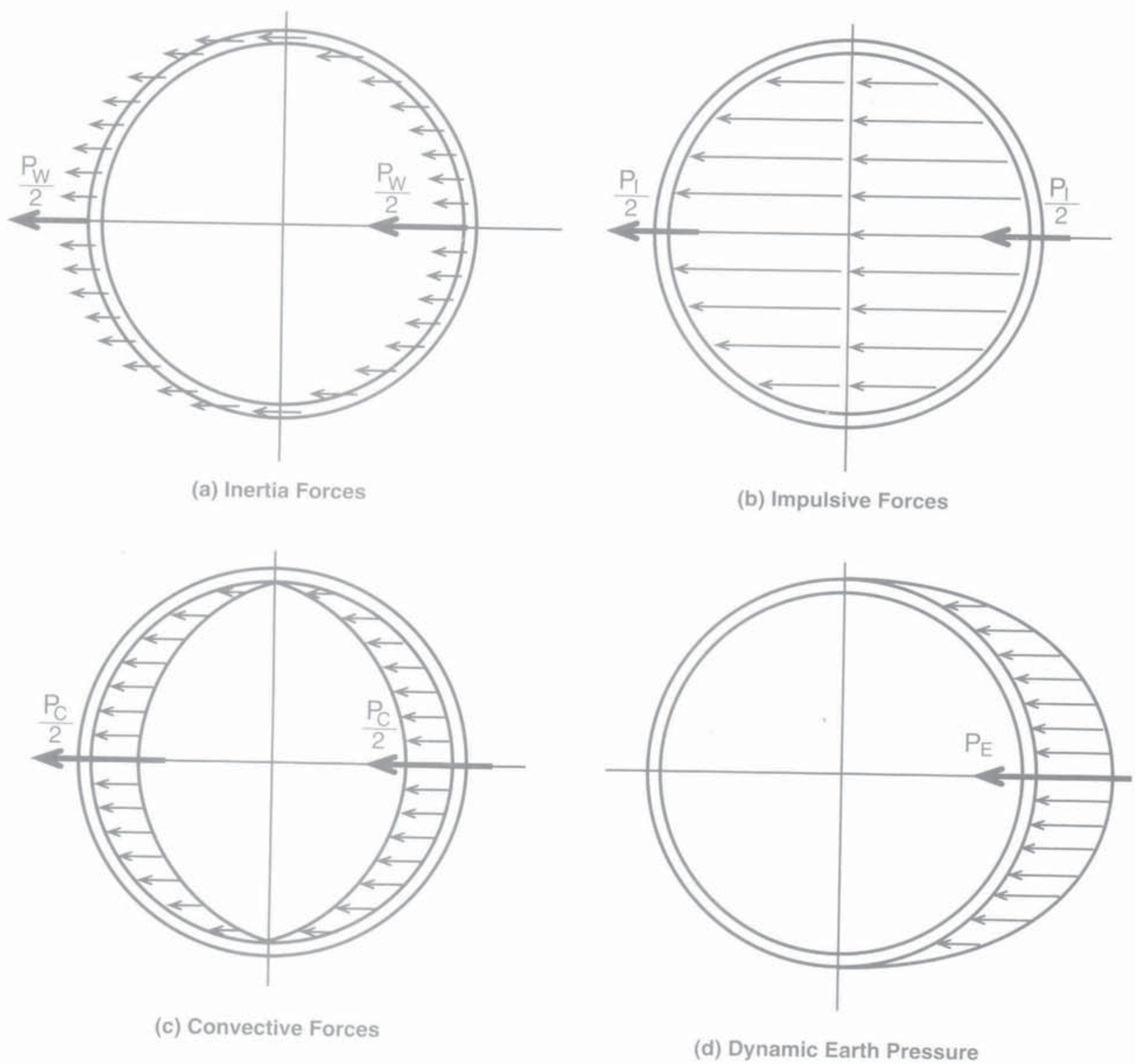


Fig. 5-7. Forces for Design of Circular Tank Walls

element (W_E) responding with the immersed element is given by

$$W_E = \pi\alpha\gamma b^2$$

where α = added weight ratio (1.25 for flat two dimensional elements vibrating normal to their axis (baffles) and 1.0 for cylindrical shapes)

γ = unit weight of liquid

b = 1/2 projected width of the element or radius of the structure at height where α is determined

The effective weight of liquid responding with the element should be added to the weight of the element and the weight of any liquid contained in the element/structure (such as in case of a center well) to determine the total hydrodynamic force on it.

The immersed elements are also subjected to a drag force because of the sloshing of the liquid. This force is directly added to the hydrodynamic force and can be determined as follows:

$$F_D = C_P A u^2$$

where F_D = force applied at the centroid of the projected area of the element

C_P = drag coefficient (use a value of 2 for plates, 1 for cylinders)

A = projected area of the submerged element

u = horizontal sloshing velocity

5.4 FOUNDATIONS

Foundations are to be designed for the combined effects of gravity and lateral forces (due to earthquake or wind) according to the applicable load combinations given in Chapter 3. Note that anchored tank foundations must be designed for the uplift force due to overturning moment of the earthquake. The base pressure is calculated by including the effect of overturning moment. The overturning moment on the tank should include the effect of liquid sloshing and hydrodynamic pressure. In case of unanchored flexible base tanks, there is no uplift on the foundation, and the tank wall may only transfer horizontal shear to the footing. In this situation, the movement of the wall relative to the foundation and the shear at contact point between the wall and the footing could control the design. The overall integrity of the tank shall be ensured by providing adequate margins of safety against both sliding and overturning of the tank.

5.5 OTHER COMPONENTS

For circular prestressed tanks with anchored flexible base, the strength of seismic cables and their anchorages in tank wall and foundation shall be investigated for tensile forces due to base shear and overturning moment.

For flexible base tanks, the strength of base pad shall be adequate for shear and compression due to combined gravity and earthquake forces. The coefficient of friction (μ) between concrete and elastomeric pad may not be taken greater than 50% of its value. The effect of contained liquid may be neglected for computation of base pad frictional resistance. The effect of vertical acceleration that reduces the frictional resistance between the base pad and concrete should be included.

For unanchored and contained flexible tanks, the strength of the containment pad, its support structure and the tank wall shall be designed for forces resulting from impulsive and convective pressure.

Where no vertical or diagonal ties are provided between walls and footing, no tension is permitted due to uplift from earthquake overturning moment. In such situations, the overturning moment should be balanced by weight and width of the structure with appropriate margin of safety.

For both anchored and unanchored and uncontained flexible base tanks, the relative displacement between the tank walls and the foundation due to combined load effect shall not exceed the radial and tangential movement capacity of the water stop to prevent leakage. Friction between the base pad and wall shall not be relied on to reduce the tangential displacements.

For flexible base tanks using flexible containment pad, the thickness of the pad and sponge if used shall not be less than 1.5 times the computed horizontal displacement of the tank base for hydrostatic and earthquake loading.

5.6 REFERENCES

5-1 *Rectangular Concrete Tanks*, Revised 5th Ed., Portland Cement Association, Skokie, IL 60077, 1998.

5-2 *Circular Concrete Tanks Without Prestressing*, Portland Cement Association, Skokie, IL 60077, 1993.

5-3 AWWA Standard for Circular Prestressed Concrete Water Tanks with Circumferential Tendons, ANSI/AWWA D115-95.

CHAPTER 6
Detailing

6.1 GENERAL

Earthquakes can induce large forces in structures that are able to remain elastic during the ground excitation. Since it is generally not feasible to design structures for such large forces, the current earthquake design philosophy allows structures to respond in the inelastic range through controlled damage and deformation at predetermined locations. The idea is for the structure to dissipate the excess earthquake energy through ductile inelastic excursions. In order for the structure to behave in this manner without collapse, it has to be detailed properly to provide the required redistribution and ductility. It is for this reason that detailing becomes an essential part of design against earthquakes.

The seismic detailing of reinforced concrete building structures is generally taken from Chapter 21 of ACI 318. The various model building codes (IBC 2000, UBC 1997, BOCA 1996, SBC 1997, UBC 1994) either refer to Chapter 21 of ACI 318 or adopt the specific provisions of ACI 318 with modifications.

The seismic detailing requirements in ACI 318 are related to the type of structural system, seismic risk level at the site, level of energy dissipation assumed in the computation of design seismic forces, and occupancy of the structure. The seismic risk levels are classified as low, moderate and high.

In order to determine the necessary detailing for a structure, the designer may have to use the applicable model building code in conjunction with the appropriate edition of ACI 318 that is referenced by the model code. The designer will determine the Seismic Zone in case of UBC, the Seismic Performance Category (SPC) in case of BOCA and SBC and Seismic Design Category (SDC) in case of IBC. These parameters are indicative of and related to the seismic risk level shown in Table 6-1. Thus Table 6-1 should be used to ascertain the appropriate seismic risk level and the coresponding level of detailing for the structure.

Table 6-2 provides a summary of the sections in Chapter 21 of ACI 318 that are applicable for earthquake design and detailing of structural components in regions of intermediate or high seismic risk for structures assigned to various seismic performance or design categories.

Because of their inherent rigidity and to prevent leakage, many liquid-containing concrete structures may be designed to remain elastic during a seismic event. The current codes allow limited inelastic action in liquid-containing structures (reflected in smaller R values for such structures) when compared to buildings. Therefore, it can be argued that codes do not expect the same amount of ductility from liquid-containing structures as they do in case of buildings. Although this would mean that stringent detailing prescribed in ACI 318 for buildings in high seismic regions should not apply to liquid-containing structures, it is prudent to provide such detailing to ensure structural performance against any unexpected events or situations.

6.2 DETAILING BASED ON ACI 318-99

Reference 6-1 gives requirements and illustrative figures on seismic detailing of reinforced concrete structural elements such as beams, columns, walls, diaphragms, slabs, footings, piles and caissons based on ACI 318-99. Some of these tables and figures illustrating design and detailing of walls are reproduced here (see Tables 6-1 through 6-5 and Figures 6-1 and 6-2). The overriding requirments of ACI 350-01 are shown wherever applicable.

6.3 REFERENCE

6-1 *Seismic Detailing of Concrete Buildings*, Portland Cement Association, Skokie, IL 60077, 2000.

Table 6-1 Seismic Risk Terminology

Code, Standard, or Resource Document	Level of Seismic Risk or Assigned Seismic Performance Category (SPC) or Seismic Design Category (SDC)		
	Low	Moderate	High
BOCA National Building Code (1993, 1996, 1999)	SPC A, B	SPC C	SPC D, E
Standard Building Code (1994, 1997, 1999)			
ASCE 7-93, 7-95			
NEHRP (1991, 1994)			
Uniform Building Code (1991, 1994, 1997)	Seismic Zone 0, 1	Seismic Zone 2	Seismic Zone 3, 4
International Building Code (2000)	SDC A, B	SDC C	SDC D, E, F
ASCE 7-98			
NEHRP (1997)			

Table 6-2 Sections of Chapter 21 to Be Satisified†

Component Resisting Earthquake Effect	Level of Seismic Risk or Assigned Seismic Performance Category (SPC) or Seismic Design Category (SDC)	
	Intermediate (21.2.1.3)	High (21.2.1.4)
Frame members	21.10	21.2–21.5
Structural walls and coupling beams	None	21.2, 21.6
Structural diaphragms and trusses	None	21.2, 21.7
Foundations	None	21.2, 21.8
Frame members not proportioned to resist forces induced by earthquake motions	None	21.2, 21.9

†In addition to requirements of Chapters 1–18 and 22.

Table 6-3 Web Reinforcement Requirements

	Sect. No.	Fig. No.
The required amounts of vertical and horizontal web reinforcement depend on the magnitude of the design shear force V_u: • For $V_u \le A_{cv}\sqrt{f'_c}$: Vertical reinf. ratio ≥ 0.0012 for No. 5 bars or smaller ≥ 0.0015 for No. 6 bars or larger Horizontal reinf. ratio ≥ 0.0020 for No. 5 bars or smaller ≥ 0.0025 for No. 6 bars or larger • For $V_u > A_{cv}\sqrt{f'_c}$: $\rho_v \ge 0.0025$ (0.003 per ACI 350-01) $\rho_n \ge 0.0025$ (0.003 per ACI 350-01) Reinforcement spacing each way shall not exceed 18 in. (12 in. per ACI 350-01) Reinforcement provided for shear strength shall be continuous and shall be distributed across the shear plane.	21.6.2.1	6-1
For $V_u > 2A_{cv}\sqrt{f'_c}$, two curtains of reinforcement must be provided.	21.6.2.2	
All continuous reinforcement in structural walls shall be anchored or spliced in accordance with the provisions for reinforcement in tension in 21.5.4.	21.6.2.3	

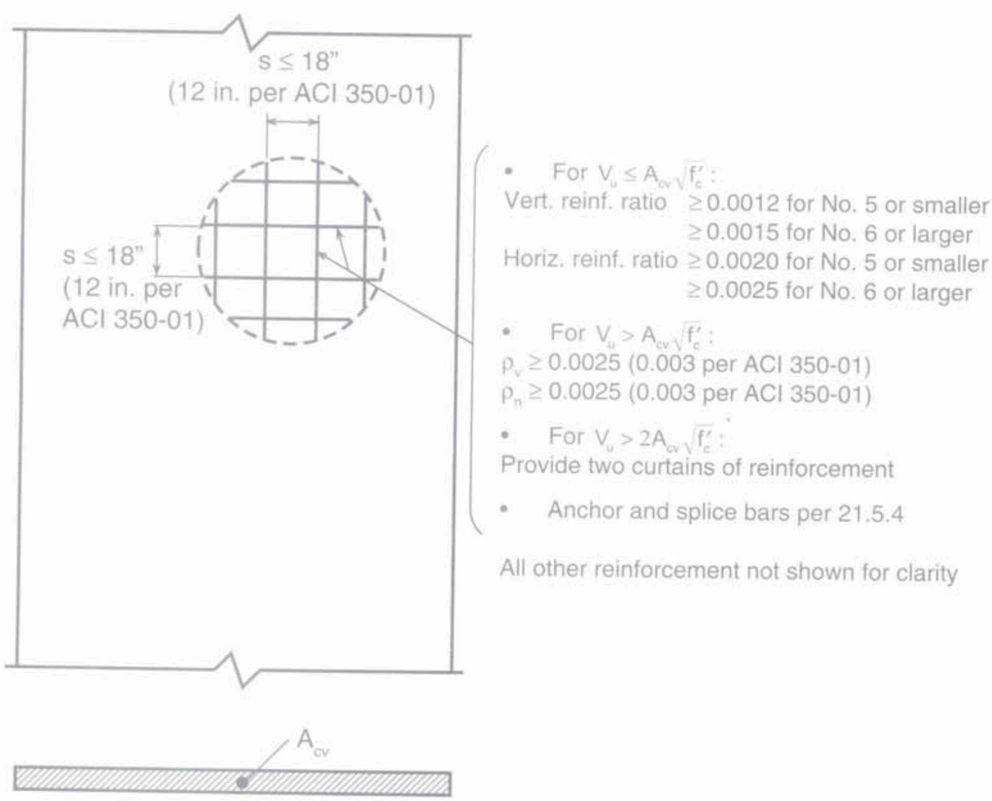


Figure 6-1 Web Reinforcement Requirements

Table 6-4 Shear Strength Requirements

	Sect. No.	Fig. No.
The nominal shear strength V_n of structural walls shall not exceed: $$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_n f_y) \qquad (21\text{-}7)$$ where α_c = 3.0 for $h_w/\ell_w \le 1.5$ = 2.0 for $h_w/\ell_w \ge 2.0$ α_c varies linearly between 3.0 and 2.0 for h_w/ℓ_w between 1.5 and 2.0.	21.6.4.1	—
In Eq. (21-7), the value of h_w/ℓ_w used for determining V_n for segments of a wall shall be the larger of the ratios for the entire wall and the segment of wall considered.	21.6.4.2	—
Walls shall have distributed shear reinforcement in two orthogonal directions in the plane of the wall. If $h_w/\ell_w \le 2.0$, $\rho_v \ge \rho_n$.	21.6.4.3	—
Nominal shear strength of all wall piers sharing a common lateral force shall not be assumed to exceed, $8A_{cv}\sqrt{f'_c}$, where A_{cv} is the total cross-sectional area, and the nominal shear strength of any one of the individual wall piers shall not be assumed to exceed $10A_{cp}\sqrt{f'_c}$, where A_{cp} is the cross-sectional area of the pier considered.	21.6.4.4	—
Nominal shear strength of horizontal wall segments and coupling beams shall be assumed not to exceed, $10A_{cp}\sqrt{f'_c}$, where A_{cp} is the cross-sectional area of a horizontal wall segment or coupling beam.	21.6.4.5	—

Table 6-5 Reinforcement Details where Boundary Elements are Not Required

	Sect. No.	Fig. No.
Where special boundary elements are not required by 21.6.6.2 or 21.6.6.3, the following shall be satisfied: • Boundary transverse reinforcement shall satisfy 21.4.4.1(c), 21.4.4.3, and 21.6.6.4(c) if the longitudinal reinforcement ratio at the wall boundary is greater than $400/f_y$. The maximum longitudinal spacing of transverse reinforcement in the boundary shall not exceed 8 in. • Horizontal wall reinforcement terminating at the ends of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement or the edge reinforcement shall be enclosed in U-stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement when $V_u \ge A_{cv}\sqrt{f'_c}$.	21.6.6.5	6-2

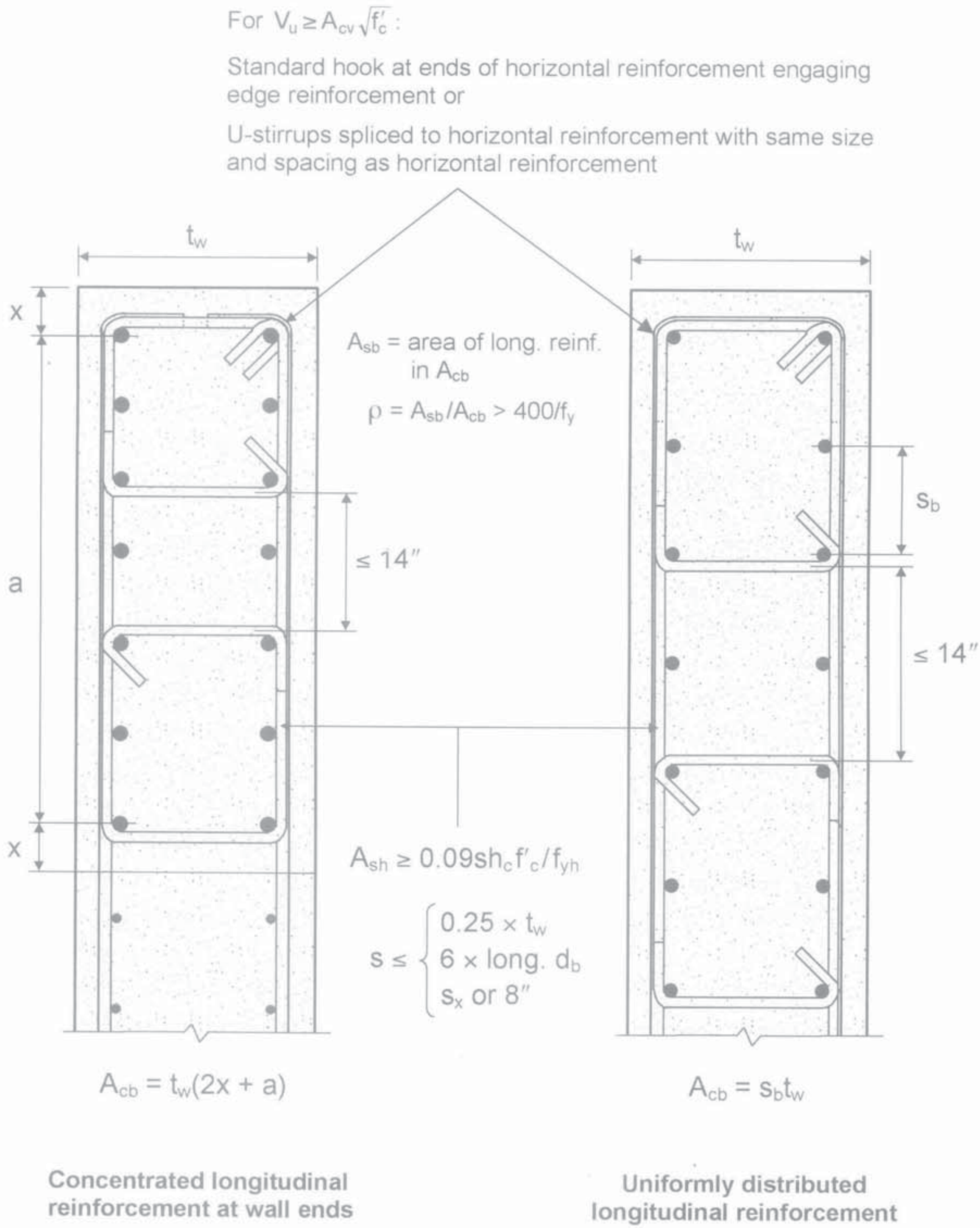


Figure 6-2 Reinforcement Details where Boundary Elements are Not Required

CHAPTER 7

Example 1: Design of Rectangular Concrete Tank

7.1 INTRODUCTION

The example rectangular tank shown in Fig. 7-1 is to be designed for earthquake forces in the N-S direction. The tank is located in the Western United States (longitude = 123°, latitude 41°) and contains non-hazardous material. The design of this tank for non-seismic load combinations is given in Ref. 5-1. The following approximate steel reinforcement was determined:

Long/short walls

Inside/Outside face – vertical	No. 5 @ 9 in.
Inside/Outside face – horizontal	No. 5 @ 12 in.

7.2 DESIGN DATA

7.2.1 General

Partially buried non-flexible base tank

Weight of contained liquid = 70 lb/ft^3

Weight of moist soil = 100 lb/ft^3

Wall thickness = 18 in.

Height of liquid = 8 ft (Ref. 5-1 conservatively assumes 10 ft of liquid height)

Concrete strength = 4,000 psi (w_c = 150 lb/ft^3, E_C = 3,834 ksi)

Reinforcement strength = 60,000 psi

7.2.2 Seismic Design Data (IBC 2000 Design)

For the given location (longitude = 123°, latitude = 41°), per IBC Section 1615.1:

$S_1 = 0.4$	(IBC Fig. 1615)
$S_S = 1.0$	(IBC Fig. 1615)
For Site Class = D	(IBC Table 1615.1.1)
$F_a = 1.1$	(IBC Table 1615.1.2(1))
$F_v = 1.6$	(IBC Table 1615.1.2(2))
Seismic coefficient $R = 2$	(IBC Table 1622.2.5(1))
Importance Factor = 1	(IBC Table 1622.2.5(2))

(This tank is not a part of a public utility facility)

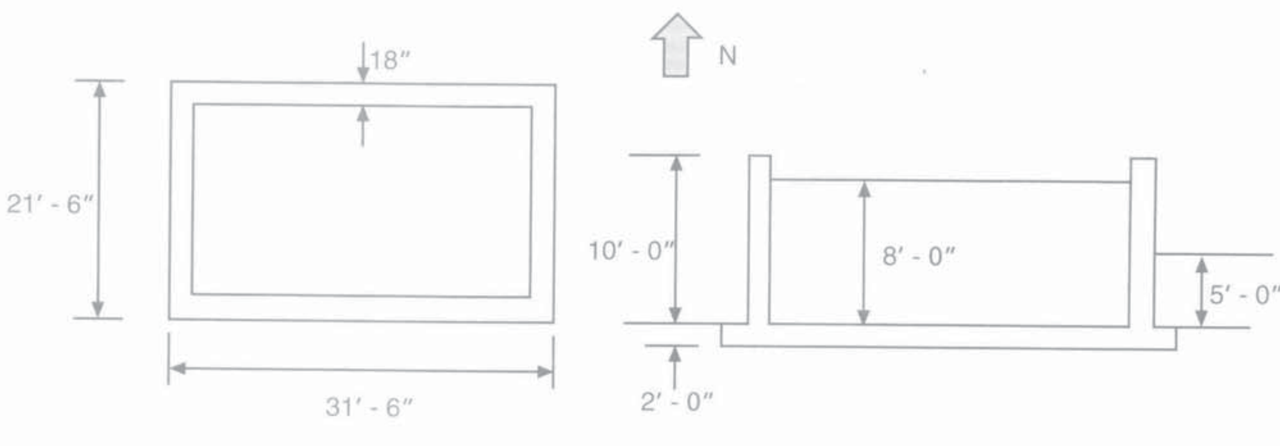


Figure 7-1 Example Rectangular Tank

7.3 SEISMIC LOAD ANALYSIS

7.3.1 Weight

$$\frac{L}{H_L} = \frac{18.5}{8} = 2.3 \qquad \text{(For analysis in N-S direction)}$$

From Fig. 4-4(a) for $\frac{L}{H_L} = 2.3$ for rectangular tanks,

$$\frac{W_I}{W_L} = 0.49 \text{ and } \frac{W_C}{W_L} = 0.51$$

$$W_L = \frac{28.5 \times 18.5 \times 8 \times 70}{1{,}000} = 295.3 \text{ kips}$$

$W_I = 0.49 \times 295.3 = 144.7$ kips

$W_C = 0.51 \times 295.3 = 150.6$ kips

$$W_W = \frac{2(20+30) \times 10 \times 1.5 \times 150}{1{,}000} = 225 \text{ kips}$$

$W_R = 0$

From Fig. 4-5, for $\frac{L}{H_L} = 2.3$, $\frac{h_I}{H_L} = 0.37$ and $\frac{h_C}{H_L} = 0.56$

$h_I = 0.37 \times 8 = 3.0$ ft

$h_C = 0.56 \times 8 = 4.5$ ft

7.3.2 Period

$$T_I = 2\pi\sqrt{\frac{W}{gK}}$$

$W = W_W + W_R + W_I = 225 + 0 + 144.7 = 369.7$ kips

$$K = \frac{E_C}{48}\left(\frac{t_w}{h}\right)^3$$

where h = mean height at which the inertia force of the tank and its contents is assumed to act.

$$h = \frac{(225 \times 5 + 144.7 \times 3)}{(225 + 144.7)} = 4.2 \text{ ft}$$

$t_w = 18$ in.

$E_c = 3{,}834$ ksi

$g = 32.2$ ft/sec^2

$$K = \frac{3834}{48}\left(\frac{18}{4.2}\right)^3 = 6{,}290 \text{ kips/ft}$$

$$T_I = 2\pi\sqrt{\frac{369.7}{32.2 \times 6290}} = 0.27 \text{sec}$$

The period associated with the convective component (T_C) can be determined as follows:

$$T_C = \frac{2\pi}{\lambda}\sqrt{L}$$

From Fig. 4-9a, $\frac{2\pi}{\lambda} = 0.66$ for $\frac{L}{H_L} = 2.3$

$T_C = 0.66\sqrt{18.5} = 2.8$ sec.
($L = 18.5$ ft for analysis in N-S direction)

7.3.3 Base Shear

$$V_I = C_{SI}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

$$V_C = C_{SC}(W_C) \qquad \text{Convective}$$

$$C_{SI} = \frac{S_{DS} I}{R} \le \frac{S_{D1} I}{R T_I}$$

$S_{MS} = F_a S_S = 1.1 \times 1.0 = 1.1$

$S_{M1} = F_v S_1 = 1.6 \times 0.4 = 0.64$

$$S_{DS} = \frac{2}{3} S_{MS} = 0.73$$

$$S_{D1} = \frac{2}{3} S_{M1} = 0.42$$

For $S_{DS} = 0.73$ and $S_{D1} = 0.42$, from Tables 1616.3(1) and 1616.3(2) of IBC 2000, Seismic Design Category SDC = D

$$C_{SI} = \frac{0.73 \times 1.0}{2} = 0.37 \le \frac{0.42 \times 1}{2 \times 0.27} = 0.78 \qquad \text{Use 0.37}$$

$$C_{SC} = \frac{S_{DS} I}{R} \le \frac{S_{D1} I}{R T_C}$$

$$C_{SC} = \frac{0.73 \times 1}{2} = 0.37 \le \frac{0.42 \times 1}{2 \times 2.8} = 0.075 \qquad \text{Use 0.075}$$

$V_I = C_{SI}(W_W + W_R + W_I)$
$= 0.37\ (225 + 0 + 144.7) = 136.8$ kips

$V_C = C_{SC}(W_C) = 0.075 \times 150.6 = 11.3$ kips

Total base shear $V_T = \sqrt{V_I^2 + V_C^2} = 137.3$ kips

7.3.4 Overturning Moment

$$M_I = C_{SI}(W_W h_W + W_R h_R + W_I h_I) \qquad \text{Impulsive}$$

$$M_C = C_{SC}(W_C h_C) \qquad \text{Convective}$$

$$M_I = C_{SI}(W_W h_W + W_R h_R + W_I h_I)$$
$$= 0.37(225 \times 5 + 0 + 144.7 \times 3) = 577 \text{ ft-kips}$$

$$M_C = C_{SC}(W_C h_C) = 0.075 \times 150.6 \times 4.5 = 50.8 \text{ ft-kips}$$

Total overturning moment $M_T = \sqrt{M_I^2 + M_C^2}$
$= 579$ ft-kips

7.3.5 Overall Stability Check

- Sliding (Neglect backfill)

Base Shear = 137 kips

Weight of tank without its contents

Walls = 225 kips

Base slab = $\dfrac{32 \times 22 \times 2 \times 150}{1{,}000} = 211$ kips

Weight of contents = 295.3 kips

Total weight = 731 kips

Coefficient of friction = 0.7

Factor of safety = $\dfrac{0.7 \times 731}{137} = 3.7$ O.K.

- Overturning

Overturning moment = 579 ft-kips

Resisting moment = $\dfrac{731 \times 22}{2} = 8{,}041$ ft-kips

Factor of safety = $\dfrac{8{,}041}{579} = 13.9$ O.K.

7.4 DESIGN OF WALLS PERPENDICULAR TO THE DIRECTION OF ANALYSIS

For determining the forces/stresses in the tank walls, the tank is divided into leading half and the trailing half portions as shown in Fig. 5-3.

Weight of long walls = $\dfrac{30 \times 10 \times 1.5 \times 150 \times 2}{1{,}000} = 135$ kips

Wall inertia,

$$P_W = \frac{S_{DS} I}{R} W_W = C_{SI} W_W = 0.37 \times 135 = 50 \text{ kips}$$

Impulsive force,

$$P_I = \frac{S_{DS} I}{R} W_I = C_{SI} W_I = 0.37 \times 144.7 = 53.4 \text{ kips}$$

Convective force,

$$P_C = \frac{S_{DS} I}{R} W_C = C_{SC} W_C = 0.075 \times 150.6 = 11.3 \text{ kips}$$

Both the leading and the trailing wall will be subjected to the combined effects of (1) wall inertia force, $\frac{P_W}{2}$ (2) one-half the impulsive force, P_I and, (3) one-half the convective force, P_C, as shown in Fig. 7-2.

$\dfrac{\Delta}{H_W} = \dfrac{0.002}{10 \times 12} = 0.000017 < 0.002$ for medium sand (see Table 4-3 and Section 7.5). Therefore, no active or passive pressure is anticipated on the walls due to negligible deformation of the tank.

The dynamic earth and ground water pressure at rest are neglected in this example (see Ref. 4-5 for detailed analysis of dynamic earth pressure).

The pressure distribution on the wall is calculated as follows (see Fig. 7-2):

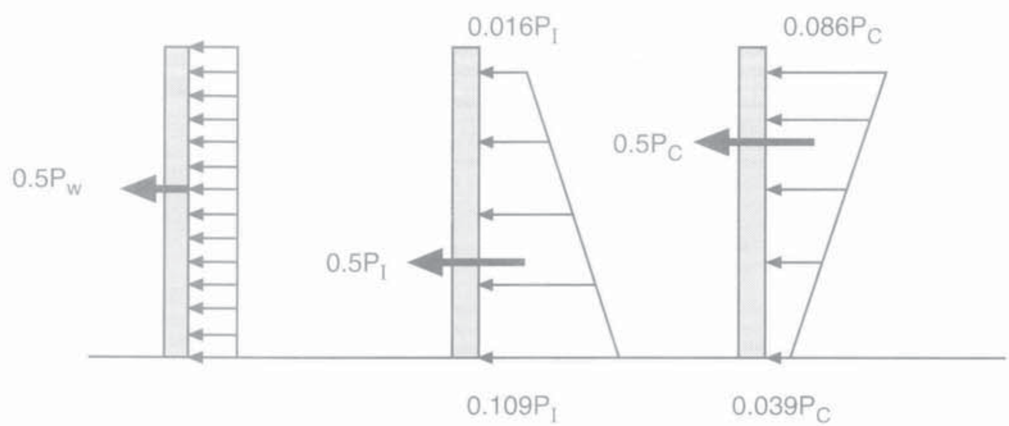


Figure 7-2 Forces on Wall Perpendicular to the Direction of Analysis

For impulsive force,

$$P_{Iy} = P_I \frac{[4H_L - 6h_I] - \left[(6H_L - 12h_I) \times {}^{y}\!/_{H_L}\right]}{2H_L^2}$$

At bottom of wall,

$$P_{Iy=0} = P_I \frac{[4\times 8 - 6\times 3] - \left[(6\times 8 - 12\times 3)\times {}^{0}\!/_{8}\right]}{2\times 8^2} = 0.109P_I$$

At top liquid level,

$$P_{Iy=8} = P_I \frac{[4\times 8 - 6\times 3] - \left[(6\times 8 - 12\times 3)\times {}^{8}\!/_{8}\right]}{2\times 8^2} = 0.0155P_I$$

For convective force,

$$P_{Cy} = P_C \frac{[4H_L - 6h_C] - \left[(6H_L - 12h_C) \times {}^{y}\!/_{H_L}\right]}{2H_L^2}$$

At bottom of wall,

$$P_{Cy=0} = P_C \frac{[4\times 8 - 6\times 4.5] - \left[(6\times 8 - 12\times 4.5)\times {}^{0}\!/_{8}\right]}{2\times 8^2} = 0.039P_C$$

At top liquid level,

$$P_{Cy=8} = P_C \frac{[4\times 8 - 6\times 4.5] - \left[(6\times 8 - 12\times 4.5)\times {}^{8}\!/_{8}\right]}{2\times 8^2} = 0.0859P_C$$

Pressure due to inertia = $\dfrac{50\times 1{,}000}{2\times 30\times 10}$ = 84 psf

Pressure due to impulsive force,

$$\text{Bottom of wall} = 0.109P_I = \frac{0.109\times 53.4\times 1{,}000}{28.5} = 204\text{ psf}$$

$$\text{Top of liquid} = 0.016P_I = \frac{0.016\times 53.4\times 1{,}000}{28.5} = 30.0\text{ psf}$$

Pressure due to convective force,

$$\text{Bottom of wall} = 0.039P_C = \frac{0.039\times 11.3\times 1{,}000}{28.5} = 15.5\text{ psf}$$

Top of liquid = $0.086P_C$ = 34 psf

Ratio of length of long wall to height of liquid

$$\frac{b}{a} = \frac{30}{8} = 3.75 \qquad \text{(Use 4.0)}$$

Ratio of length of short wall to height of liquid

$$\frac{c}{a} = \frac{20}{8} = 2.50$$

The moment, shear and deflection due to the above forces are determined using Ref. 5-1. The moment and deflection coefficients are taken from Chapter 3 and the shear coefficients are taken from Chapter 2 of this reference for the specific loading and end conditions of the walls. The coefficients for the long wall are determined for $\frac{b}{a} = 4$ and $\frac{c}{a} = 2.5$ by interpolation, as follows:

Long wall subjected to triangular load

M_x coefficient for vertical steel	= 150
M_y coefficient for horizontal steel	= 88
Deflection coefficient	= 26
Shear coefficient—bottom	= 0.5
Shear coefficient—side	= 0.38

Long wall subjected to uniform load

M_x coefficient for vertical steel	= 435
M_y coefficient for horizontal steel	= 348
Deflection coefficient	= 98
Shear coefficient—bottom	= 1.0
Shear coefficient—side	= 1.68

Design moment per IBC Eq. 16-5, See Chapter 3,

$U = 1.2D + 1.0E + 1.2F$

$= 0.9D + 1.0E + 1.2F$

E corresponds to the effects of earthquake force computed in Table 7-1. F corresponds to the effects of static hydrostatic fluid pressure computed in Ref. 5-1.

Maximum hydrodynamic moments from Table 7-1,

Vertical direction,
M_{dx} = 6.2 ft-kips (74.4 in.-kips) per lineal foot

Horizontal direction,
M_{dy} = 4.6 ft-kips (55.2 in.-kips) per lineal foot

Maximum hydrostatic moments from Ref. 5-1,

Vertical direction,
M_{hx} = 108.6 in.-kips per lineal foot

Horizontal direction,
M_{hy} = 65.7 in.-kips per lineal foot

Table 7-1 Design Forces on Long Wall due to Earthquake Forces

Loading	Inertia	Impulsive			Convective			SRSS[2]
Pressure, q (psf)								
	Uniform	Uniform + Triangular			Uniform - Triangular			
Top	84	30	0.0	30	34	0.0	34	
Bot.	84	30	174	204	34	-18.5	15.5	
Height, h (ft)	10	8	8	8	8	8	8	
Design Coefficients[1]								
M_x Coeff.	435	435	150		435	150		
M_y Coeff.	348	348	88		348	88		
Shear Coeff.								
-Bottom	1.0	1.0	0.5		1.0	0.5		
-Side	1.7	1.7	0.4		1.7	0.4		
Deflection Coeff.	98.0	98.0	26.0		98.0	26.0		
Wall Forces and Deformations								
M_{dx} (ft-kips)	3.65	0.84	1.67	2.51	0.97	-0.18	0.79	6.2
M_{dy} (ft-kips)	2.92	0.66	0.98	1.64	0.78	-0.11	0.67	4.6
Shear (k)								
- Bot.	0.84	0.24	0.70	0.94	0.28	-0.08	0.20	1.8
- Side	1.43	0.41	0.56	0.97	0.48	-0.06	0.42	2.4
Deflection (in.)	0.0061	0.00089	0.0014	0.0023	0.0010	-0.00015	0.0009	0.009

[1] Design coefficients are taken from Ref. 5-1 for different loading patterns and end conditions of the plates.

$M_{dx} = M_x$ *coefficient* × *pressure* × *height*^{2}/1,000

$M_{dy} = M_y$ *coefficient* × *pressure* × *height*^{2}/1,000

Shear = Shear coefficient × pressure × height

Deflection = Deflection coefficient × pressure × height^4/1,000D, *where* $D = E_c t^3/12(1-\mu^2)$ = 1,940,962.5 *in.-kips, and* $\mu = 0.2$.

[2] Note that convective force is out-of-phase with both inertia and impulsive forces. Therefore, square root of the sum of squares (SRSS) method is used to combine the inertia and impulsive forces with the convective forces.

Total moments using the above load combination,

Vertical direction,

$M_{ux} = 1.2 \times 0 + 1 \times 74.4 + 1.2 \times 108.6 = 204.7$ in.-kips

Horizontal direction,

$M_{uy} = 1.2 \times 0 + 1 \times 55.2 + 1.2 \times 65.7 = 134.0$ in.-kips

Using ACI 318 load combination,

$U = 0.75(1.4D + 1.7L + 1.7^* F + 1.87E/1.4)$

$= 1.28F + 1.0E$

$M_{ux} = 1.3 \times 108.6 + 74.4 = 215.6$ in.-kips

$M_{uy} = 1.3 \times 65.7 + 55.2 = 140.6$ in.-kips

Steel provided (No. 5 @ 9 in.) is sufficient to take care of $M_{ux} = 215.6$ in.-kips in the vertical direction.

Steel provided (No. 5 @ 12 in.) is sufficient to take care of $M_{uy} = 140.6$ in.-kips in the horizontal direction.

*See Section 3.7.2 (Modification 1)

7.5 DESIGN OF WALLS PARALLEL TO THE DIRECTION OF ANALYSIS

It is assumed that the total base shear will be equally resisted by the two walls.

$$\text{Shear in each wall} = \frac{137.3}{2} = 69 \text{ kips}$$

Required strength per IBC 2000, (see Chapter 3)

$$U = 1.2D + 1.0E + 1.2F = 1.0E$$
$$= 0.9D + 1.0E + 1.2F = 1.0E$$
$$V_u = 1.0 \times 69 = 69 \text{ kips}$$

Nominal shear strength per ACI 318-99

$$V_n = A_{CV}(\alpha_C \sqrt{f'_c} + \rho_n f_y)$$

$$\alpha_c = 3 \text{ for } \frac{h_w}{\ell_w} = 0.5$$

$$\text{For No. 5 @ 12 in. on each face, } \rho_n = \frac{2 \times 0.31}{(18 \times 12)} = 0.00287$$

$$\phi V_n = \frac{0.85 \times 21.5 \times 18 \times 12(3\sqrt{4,000} + 0.00287 \times 60,000)}{1,000}$$

$$= 1428.7 \text{ kips} >> 69 \text{ kips} \qquad \text{OK}$$

Per ACI 350-01, the steel reinforcement required each way each face is 0.003. Therefore, ρ_n should be increased to No. 5 @ 10 in.

Since $\frac{h_w}{\ell_w}$ is less than 2, $\rho_v = \rho_n$. Therefore, No.5 @ 9 in. is more than adequate in the vertical direction.

The steel ratio provided is more than 0.0012 in the vertical direction and 0.0020 in the horizontal direction as required for the condition $V_u < A_{CV}\sqrt{f'_c}$ (ACI 21.6.2.1 and 14.3, Table 6-3). Also, the spacing provided meets the 18 in. maximum spacing requirement per ACI 318 and the 12 in. maximum spacing requirement per ACI 350.

Wall deformation,

$$\text{Flexural stiffness} = \frac{3E_C I}{H_W^3} = \frac{3 \times 3,834 \times 1.5 \times 20^3}{10^3}$$
$$= 138,024 \text{ kips/in.}$$

$$\text{Shear stiffness} = \frac{GA}{1.2H_W} = \frac{0.4 \times 3,834 \times 1.5 \times 20 \times 12}{1.2 \times 10}$$
$$= 46,008 \text{ kips/in.}$$

$$\text{Flexural deformation} = \frac{69}{138,024} = 0.0005 \text{ in.}$$

$$\text{Shear deformation} = \frac{69}{46,008} = 0.0015 \text{ in.}$$

Total deformation = 0.002 in.

7.6 DETAILING

The seismic forces and the corresponding reinforcement requirements in this example are small compared to the requirements under other load and serviceability conditions. This indicates that this particular tank is likely to remain nearly elastic or distress free in the event of a design earthquake.

However, based on the SDC *D* associated to this structure, detailing corresponding to high seismic risk will apply per Table 6-1 (see Chapter 6). Based on this, it is prudent to locate the splices away from the potential plastic hinge zones near the bottom of the walls (Figure 7-3). The Class B splice length for No. 5 bars per 12.2 of ACI 318-99 is 18.5 in. The required development length of the No. 5 dowels in the base slab and foundation is 14 in. These computations are shown in Ref. 5-1.

The requirements of Table 6-3 are satisfied for in-plane wall design. The steel provided along with spacing satisfies Section 21.6.2.1 of ACI 318 (Table 6-3). Per Section 21.6.2.3, all continuous reinforcement in structural walls should be anchored or spliced in accordance with the provisions of reinforcement in tension (21.5.4).

Since there is negligible axial load on the walls (less than $0.2f'_c$), boundary elements are not required (21.6.6.2 or 21.6.6.3). Boundary transverse reinforcement shall satisfy 21.4.4.1(c), 21.4.4.3 and 21.6.6.4(c) if the longitudinal reinforcement ratio at the wall boundary is less than $400/f_y$ (Table 6-5).

Longitudinal reinforcement ratio,

$$\rho = \frac{0.31 \times 2 \times 12}{9 \times 18 \times 12} = 0.0038$$

$$\frac{400}{f_y} = \frac{400}{60,000} = 0.00667 > 0.0038$$

Therefore, transverse reinforcement requirements stipulated above (Table 6-5) do not apply.

Where $V_u > A_{CV}\sqrt{f'_c}$, the horizontal reinforcement terminating at the ends of structural walls without boundary elements shall have a standard hook engaging the edge reinforcement (Table 6-5, Section 21.6.6.5). Since $V_u < A_{CV}\sqrt{f'_c}$, this detail is not required. However, it is recommended that this detail be considered so that the reinforcement is effective in resisting shear forces and the potential of buckling of vertical edge reinforcement is minimized.

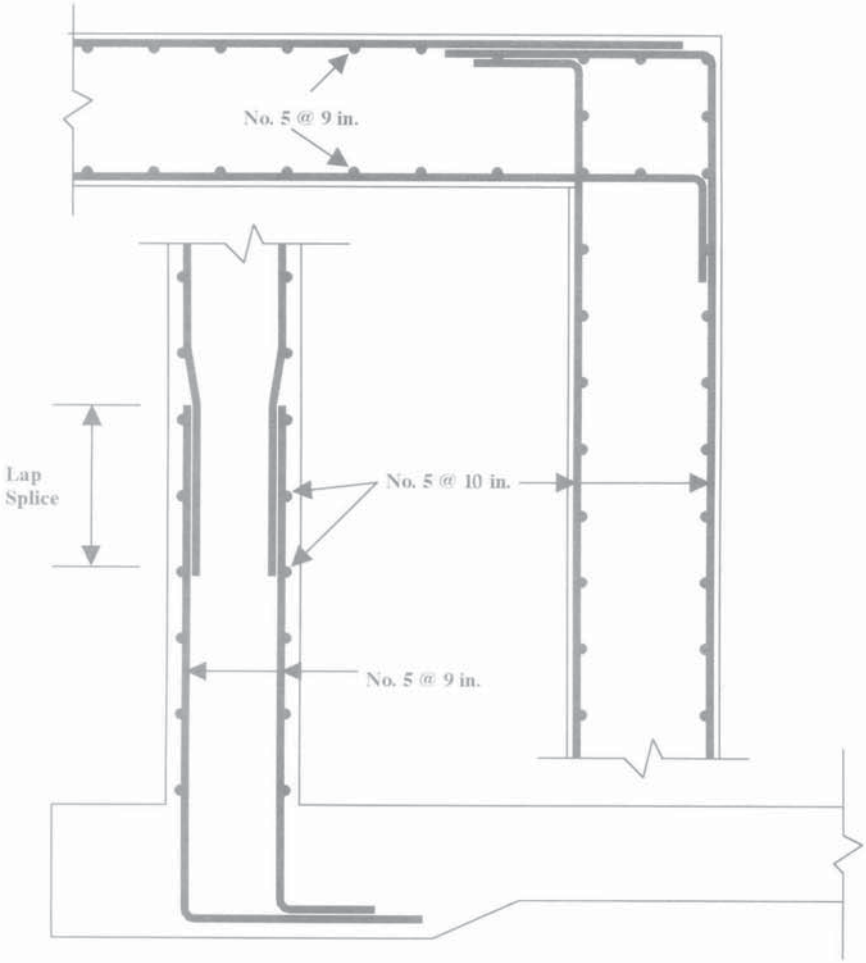


Figure 7-3 Detailing of Wall

7.7 BASE SHEAR USING UBC 1997

For the tank located in Seismic Zone 4, $Z = 0.4$ (Table 16-I)

Seismic source type = B with site located 10 km from a known seismic source (Tables 16-T and 16-U)

Soil profile type = S_D (Table 16-J)

Seismic importance factor = 1 (Non hazardous material)

Response modification factor, $R = 2.9$ (Table 16-P)

For Z = 0.4 and soil profile type D,

$$C_a = 0.44N_a \quad \text{Table 16-Q}$$

$$C_v = 0.64\,N_v \quad \text{Table 16-R}$$

From Tables 16-S and 16-T, the near-source factors $N_a = 1$ and $N_v = 1$, respectively.

$$C_a = 0.44N_a = 0.44$$

$$C_v = 0.64N_v = 0.64$$

Using Section 1634.3 for flat-bottom rigid tanks,

$$V = 0.7C_a IW$$

$$= 0.7 \times 0.44 \times 1 \times 520.3 = 160.3 \text{ kips}$$

Note that W includes the weight of tank and contained liquid.

As indicated in Section 3.2.2 of Chapter 3, UBC 1997 allows the use of alternate procedure such as the one given in ACI 350.3. The guidelines for use of this method in conjunction with UBC 1997 are given in Chapter 4 (See Section 4.3).

Base shear,

$$V_i = \frac{C_V I}{RT_i}(W_W + W_R + W_i) \quad \text{Impulsive}$$

$$V_i = \frac{0.64 \times 1}{2.9 \times 0.27}(225 + 0 + 144.7) = 302.2 \text{ kips}$$

In the short period range, the impulsive base shear shall be limited by,

$$V_I = \frac{2.5C_a I}{R}(W_W + W_R + W_I) = 140.4 \text{ kips} < 302.2 \text{ kips},$$

Use $V_I = 140.4$ kips

$$V_C = \frac{C_V I}{RT_C}(W_C) \qquad \text{Convective}$$

$$V_C = \frac{1 \times 0.64 \times 1}{2.9 \times 2.8}(150.6) = 11.8 \text{ kips}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2} = 140.9$ kips

7.8 BASE SHEAR USING UBC 1994

The provisions of ACI 350.3 were essentially developed to be compatible with UBC 1994 as discussed in Chapters 3 and 4. Therefore, no interpretations or extensions were made in application of ACI 350.3 with this building code.

For the tank located in Seismic Zone 4, $Z = 0.4$ (Table 16-I)

Site coefficient, $S = 1.5$ (Table 16-J)

Soil profile type $= S_3$ (Table 16-J)

Seismic importance factor = 1
(Table 4-1, Non hazardous material)

Response modification factor, $R_W = 2.75$
(Table 4-2)

Base shear,

$$V_I = \frac{ZIC_I}{R_W}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

$$V_C = \frac{ZIC_C}{R_W}(W_C) \qquad \text{Convective}$$

$$C_I = 2.75 \text{ for } T_I < T_S \qquad \text{Impulsive}$$

$$= \frac{1.25S}{T_I^{2/3}} \text{ for } T_I > T_S$$

$$C_C = \frac{1.25S}{T_C^{2/3}} \qquad \text{Convective}$$

T_S can be conservatively taken as 0.31 s.

Since $T_I < T_S$, $C_I = 2.75$

$$C_C = \frac{1.25 \times 1.5}{2.8^{2/3}} = 0.94$$

$$V_I = \frac{0.4 \times 1 \times 2.75}{2.75}(225 + 0 + 144.7) = 147.9 \text{ kips}$$

$$0.075 ZIW_T = 0.075 \times 0.41 \times 520.3 = 15.6 \text{ kips} < 149.3 \text{ kips}$$

$$V_C = \frac{0.4 \times 1 \times 0.94}{2.75}(150.6) = 20.6 \text{ kips}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2} = 149.3$ kips

Note that ACI 350.3 recommends using $C_C = \frac{6S}{T_C^2}$ for

$T_C \geq 2.4$ sec and $R_{WC} = 1$ for convective motion.

$$C_C = \frac{6 \times 1.5}{2.8^2} = 1.15$$

$$V_C = \frac{0.4 \times 1 \times 1.15}{1.0}(150.6) = 69.2 \text{ kips}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2} = 163.3$ kips

7.9 BASE SHEAR USING BOCA AND SBC

Seismic Hazard Exposure Group = I

Seismic Coefficients,

$A_a = 0.2$

$A_v = 0.3$

Based on $A_v = 0.3$ and Seismic hazard exposure group = I, Seismic performance category SPC = D

Site coefficient = 1.5 (Soil Profile Type S_3)

Response Modification Factor $R = 3$ (Table 9.2.7.5, ASCE 7-95)

$$V_I = C_{SI}(W_W + W_R + W_I) \qquad \text{Impulsive}$$

$$V_C = C_{SC}(W_C) \qquad \text{Convective}$$

$$C_{SI} = \frac{1.2 A_V S}{RT_I^{2/3}} \leq \frac{2.5 A_a}{R}$$

$$C_{SI} = \frac{1.2 \times 0.3 \times 1.5}{3 \times 0.27^{2/3}} = 0.43$$

$$\frac{2.5 A_a}{R} = \frac{2.5 \times 0.3}{3} = 0.25$$

Use $C_{SI} = 0.25$

$$C_{SC} = \frac{1.2A_V S}{RT_C^{2/3}} = \frac{1.2 \times 0.3 \times 1.5}{3 \times 2.8^{2/3}} = 0.09$$

$V_I = 0.43(225 + 0 + 144.7) = 159$ kips

$V_C = 0.09(150.6) = 13.6$ kips

$V_T = \sqrt{V_I^2 + V_C^2} = 159.6$ kips

7.10 BASE SHEAR COMPARISON

Code	Base Shear (kips)	
IBC 2000	137.3	
UBC 1997	140.9	
UBC 1994	149.3*	Service Level
ACI 350.3	163.3*	Service Level
BOCA 1996/SBC 1997	159.6	

*Note that V_T = 149.3 kips and 163.3 kips are service level base shears that are multiplied by a factor of 1.4 when combined using the load combinations given in Chapter 3 to determine the design force in a member. The base shears computed for IBC 2000, UBC 1997 and SBC/BOCA are strength level and are multiplied by a factor of 1.0 in the load combinations.

CHAPTER 8

Example 2: Design of Circular Concrete Tank

8.1 INTRODUCTION

The example circular tank shown in Fig. 8-1 is to be designed for earthquake forces. The tank is located in the Western United States (longitude = 123°, latitude 41°) and contains non-hazardous material. The design for non-seismic load combinations of this tank given in Ref. 5-2 results in the following steel reinforcement:

- Vertical reinforcement

Inside face	No. 6 @ 8 in. (Bottom 15 ft) No. 5 @ 8 in. (Top 13 ft)
Outside face	No. 6 @ 12 in.

- Horizontal reinforcement

Inside/Outside face	No. 9 @ 8 in. (Bottom 15 ft)
	No. 8 @ 8 in. (Top 13 ft)

8.2 DESIGN DATA

8.2.1 General

Partially buried non-flexible base tank with rigid base and hinged connection between the wall and the roof slab.

Weight of contained liquid = 65 lb/ft^3

Weight of moist soil = 90 lb/ft^3

Wall thickness = 16 in.

Height of liquid = 26 ft

Concrete strength = 4,000 psi
(w_c = 150 lb/ft^3, E_c = 3,834 ksi)

Reinforcement strength = 60,000 psi

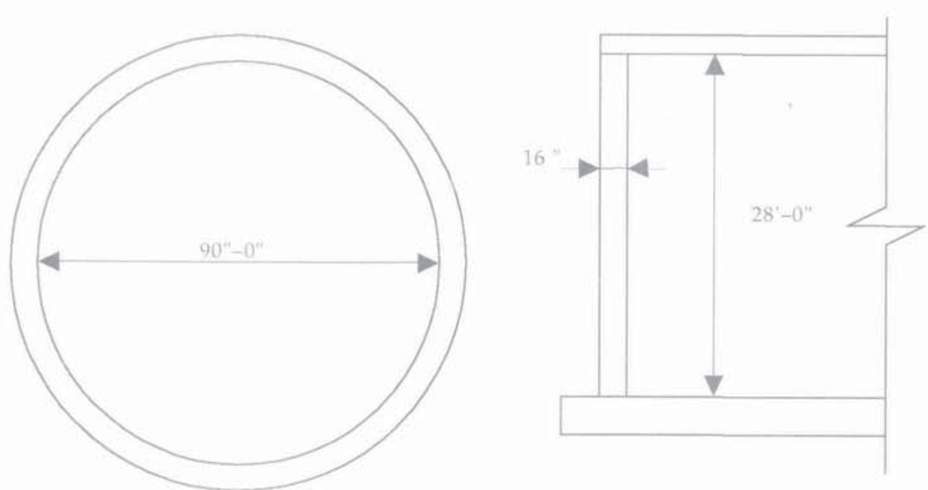


Figure 8-1 Example Circular Tank

8.2.2 Seismic Design Data (IBC 2000 Design)

For the given location (longitude = 123°, latitude = 41°), per IBC Section 1615.1:

$S_1 = 0.4$	(IBC Fig. 1615)
$S_s = 1.0$	(IBC Fig. 1615)
For Site Class = D	(IBC Table 1615.1.1)
$F_a = 1.1$	(IBC Table 1615.1.2)
$F_v = 1.6$	(IBC Table 1615.1.2)
Seismic coefficient $R = 2$	(IBC Table 1622.2.5(1))
Importance factor = 1	(IBC Table 1622.2.5(2))

(This tank is not a part of public utility facility)

8.3 SEISMIC LOAD ANALYSIS

8.3.1 Weight

$$\frac{D}{H_L} = \frac{90}{26} = 3.46$$

From Fig. 4-4(b) for $\frac{D}{H_L}$ = 3.5 for circular tanks,

$$\frac{W_I}{W_L} = 0.35 \text{ and } \frac{W_C}{W_L} = 0.65$$

$$W_L = \frac{3.14 \times 90^2 \times 26 \times 65}{4 \times 1{,}000} = 10{,}751 \text{ kips}$$

$$W_I = 0.35 \times 10{,}751 = 3{,}763 \text{ kips}$$

$$W_C = 0.65 \times 10{,}751 = 6{,}988 \text{ kips}$$

$$W_W = \frac{3.14 \times (92.67^2 - 90^2) \times 28 \times 150}{4 \times 1{,}000} = 1{,}609 \text{ kips}$$

$$W_R = \frac{3.14 \times 92.67^2 \times 1 \times 150}{4 \times 1{,}000} = 1{,}012 \text{ kips}$$

(assume 1 ft thickness of slab)

The roof is supported by 12 interior columns which help carry the slab vertical load. The vertical load transferred to the walls is small. Note that half of the column weight should be added to the roof weight for lateral load analysis.

The column weight (assume 2 ft diameter columns) will add another 80 kips to the roof weight. Therefore, total roof weight = 1,012 + 80 = 1,092 kips.

8.3.2 Period

For non sliding base (Section 4.7.2),

$$T_I = \frac{2\pi}{\omega_I}$$

$$\omega_I = C_L \frac{12}{H_L} \sqrt{\frac{E_C}{\rho_C}}$$

$$C_L = 10 C_W \sqrt{\frac{t_w}{12r}}$$

ρ_c = mass density of concrete (4.66 lb-sec^2 / ft^4), t_w = 16 in., R = 45 ft, $C_W = 0.143$ for $\frac{D}{H_L} = 3.5$ (Fig. 4-10).

$$C_L = 10 \times 0.143 \sqrt{\frac{16}{12 \times 45}} = 0.25$$

$$\omega_I = 0.25 \times \frac{12}{26} \sqrt{\frac{3{,}834 \times 1{,}000}{4.66}} = 104.7 \text{ rad/sec}$$

$$T_I = \frac{2\pi}{104.7} = 0.06 \text{ sec}$$

The period associated with the convective component (T_C) can be determined as follows:

$$T_C = \frac{2\pi}{\lambda}\sqrt{D}$$

From Fig. 4-9(b), $\frac{2\pi}{\lambda} = 0.65$ for $\frac{D}{H_L} = 3.5$

$$T_C = 0.65\sqrt{90} = 6.2 \text{ sec}$$

8.3.3 Base Shear

$$V_I = C_{SI}\left(W_W + W_R + W_I\right) \qquad \text{Impulsive}$$

$$V_C = C_{SC}\left(W_C\right) \qquad \text{Convective}$$

$$C_{SI} = \frac{S_{DS} I}{R} \le \frac{S_{D1} I}{R T_I}$$

$$S_{MS} = F_a S_s = 1.1 \times 1.0 = 1.1$$

$$S_{M1} = F_v S_1 = 1.6 \times 0.4 = 0.64$$

$$S_{DS} = \frac{2}{3} S_{MS} = 0.73$$

$$S_{D1} = \frac{2}{3} S_{M1} = 0.43$$

For $S_{DS} = 0.73$ and $S_{D1} = 0.43$, from Tables 1616.3(1) and 1616.3(2) of IBC 2000, Seismic Design Category SDC = *D*

$$C_{SI} = \frac{0.73 \times 1.0}{2} = 0.37 \leq \frac{0.43 \times 1}{2 \times 0.06} = 3.58 \text{ Use } 0.37$$

$$C_{SC} = \frac{S_{DS} I}{R} \leq \frac{S_{D1} I}{R T_C}$$

$$C_{SC} = \frac{0.73 \times 1}{2} = 0.37 \leq \frac{0.43 \times 1}{2 \times 6.2} = 0.035 \text{ Use } 0.035$$

$$V_I = C_{SI}(W_W + W_R + W_I)$$
$$= 0.37\ (1{,}609 + 1{,}092 + 3{,}763) = 2{,}392 \text{ kips}$$

$$V_C = C_{SC}(W_C) = 0.035 \times 6{,}988 = 245 \text{ kips}$$

Total base shear $V_T = \sqrt{V_I^2 + V_C^2} = 2{,}405$ kips

Since $T_C > 4$ sec., (See section 4.2.3)

$$S_{aC} = \frac{6 S_{D1}}{T_C^2} = \frac{6 \times 0.43}{6.2^2} = 0.067$$

With $R_{WC} = 1$ for convective motion per ACI 350.3 (Table 4-2),

$$C_{SC} = \frac{0.067 \times 1}{1} = 0.067 > 0.035$$

$$V_C = C_{SC}(W_C) = 0.067 \times 6{,}988 = 468 \text{ kips}$$

Total base shear per ACI 350.3,

$$V_T = \sqrt{V_I^2 + V_C^2} = 2{,}437 \text{ kips}$$

Using Response Spectrum method (Section 4.2.3),

$$T_S = \frac{S_{D1}}{S_{DS}} = 0.59 \text{ sec}$$

$$T_0 = 0.2 T_S = 0.118 \text{ sec}$$

Since $T_I < T_0$, $S_a = S_{DS}\left[\frac{0.6T}{T_0} + 0.4\right]$

$$S_a = 0.73 \times \left[\frac{0.6 \times 0.06}{0.118} + 0.4\right] = 0.51$$

$$C_{SI} = \frac{S_{aI}}{R} = \frac{0.51}{2} = 0.26 < 0.37$$

$$V_I = C_{SI}(W_W + W_R + W_I)$$
$$= 0.26\ (1{,}609 + 1{,}092 + 3{,}763) = 1{,}681 \text{ kips}$$

$V_C = 468$ kips

$V_T = \sqrt{(1{,}681)^2 + (468)^2}$ 1,745 kips

80% of 2,437 = 1,950 kips > 1,745 kips

Use $V_T = 1{,}950$ kips

8.3.4 Overturning Moment

$$M_I = C_{SI}(W_W h_W + W_R h_R + W_I h_I) \qquad \text{Impulsive}$$
$$M_C = C_{SC}(W_C h_C) \qquad \text{Convective}$$

From Fig. 4-5(b), for $\frac{D}{H_L} = 3.5$, $\frac{h_I}{H_L} = 0.375$ and $\frac{h_C}{H_L} = 0.54$

$h_I = 0.375 \times 26 = 9.75$ ft

$h_C = 0.54 \times 26 = 14.0$ ft

$$M_I = C_{SI}(W_W h_W + W_R h_R + W_I h_I)$$
$$= 0.37[(1{,}609 \times 14) + (1{,}092 \times 28) + (3{,}763 \times 9.75)]$$
$$= 33{,}223 \text{ ft-kips}$$

$$M_C = C_{SC}(W_C h_C) = 0.035 \times 6{,}988 \times 14 = 3{,}424 \text{ ft-kips}$$

$$M_T = \sqrt{M_I^2 + M_C^2} = 33{,}400 \text{ ft-kips}$$

Per ACI 350.3, $M_C = 0.067 \times 6{,}988 \times 14 = 6{,}555$ ft-kips

$M_T = 33{,}864$ ft-kips

For simplicity, the base shear and overturning moment computed using IBC 2000 without ACI 350.3 modifications will be used for rest of design.

8.3.5 Overall Stability Check

- Sliding (Neglecting backfill)

Weight of tank without its contents

Walls	= 1,609 kips
Roof	= 1,012 kips
Columns	= 160 kips
Base slab = 3.14 x 96.67² x 2 x 150/(4 x 1000)	= 2,202 kips
Contents = 3.14 x 90² x 26 x 65/4000	= 10,751 kips
Total weight	=15,734 kips

Neglect soil weight on footing overhang

Coefficient of friction = 0.7

Base shear = 2,405 kips

$$\text{Factor of safety} = \frac{0.7 \times 15{,}734}{2{,}405} = 4.6 \quad \text{O.K.}$$

- Overturning

Overturning moment = 33,400 ft-kips

Resisting moment = 15,734 x 45 = 708,030 ft-kips

$$\text{Factor of safety} = \frac{708,030}{33,400} = 21.2 \quad \text{O.K.}$$

8.4 DESIGN OF WALLS

The earthquake base shear is transmitted partially by membrane (tangential) shear and partially by radial shear that causes vertical bending. ACI 350.3 indicates that 80% of the base shear can be assumed to be transferred through tangential shear for tanks with D/H of 4. The remaining 20% will be transferred through vertical bending. The maximum tangential shear occurs at a point on tank wall oriented 90 degrees from the direction of earthquake, as shown in Fig. 8-2.

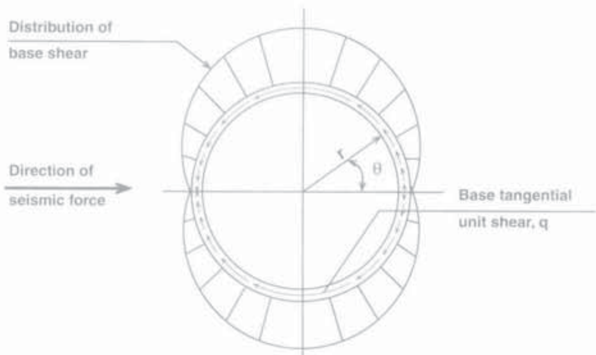


Figure 8-2 Shear transfer at base

Per IBC 2000 (see Chapter 3),

Required strength $U = 1.2D + 1.0E + 1.2F$ (see Chapter 3, Table 3-1)

Per ACI 318 and ACI 350 (see Chapter 3),

Required strength, $U = 0.75(1.4D + 1.7L + 1.7^{*} F + 1.87E/1.4)$

8.4.1 Design of Walls for In-Plane Loading

Since the effect of both D and F are negligible for in-plane wall design, required strength is $U = 1.0E$ for both IBC and ACI 318 load combinations. The shear stress from hydrostatic forces acts in a radial direction, and is not added to the in-plane shear of the wall.

Shear $V_u = 1.0E = 2{,}405$ kips (in-plane)

Maximum shear stress per ACI 350.3,

$$V_c = \frac{0.8V}{\pi r} = \frac{0.8 \times 2,405}{3.14 \times 45} = 13.6 \text{ kips/ft}$$

Nominal shear strength = $A_{CV}(\alpha_C \sqrt{f'_c} + \rho_n f_y)$
Eq (21-7), ACI 318-99

Use $\alpha_c = 3$ for $\frac{h_w}{\ell_w} < 2$

For No. 9 @ 8 in. on each face,

$$\rho_n = \frac{2 \times 1.0 \times 12}{8 \times 16 \times 12} = 0.0156$$

$$\phi V_n = 0.85 \times 12 \times 16[(3 \times \sqrt{4,000}) + (0.0156 \times 60,000)]/1,000$$

$$= 183.7 \text{ kips/ft} >> 13.6 \text{ kips/ft} \quad \text{O.K.}$$

No.6 @ 8 in. is also more than adequate in the vertical direction.

The steel ratio provided is more than 0.0012 in the vertical direction and 0.002 in the horizontal direction as required for the condition $V_u < A_{CV}\sqrt{f'_c}$ (ACI 21.6.2.1). Also, the spacing provided meets the 18 in. minimum requirement. The reinforcement provided also satisfies ACI 350 requirements ($\rho_{min} = 0.003$ and maximin spacing = 12 in.)

8.4.2 Design of Walls for Out-of-Plane Loading

For the portion of walls loaded perpendicular to their plane, the effects of earthquake forces and hydrostatic forces will be combined based on the most critical load combinations:

$$U = 0.75(1.4D + 1.7L + 1.7F + 1.87E/1.4)$$

$$= 1.3F + 1.0E$$

Moment $M_u = 1.3M_F + 1.0M_E$

From hydrostatic loading, M_F = 10,416 ft-lb/ft (Ref. 5-2). Note that this reference assumes full height of contained liquid.

For determining the out-of-plane moment due to earthquake forces M_E, the tank is divided into leading half and the trailing half portions as shown in Fig. 5-6.

As discussed in Chapter 5, the cylindrical walls (Fig. 5-7) are designed for (a) wall inertia distributed uniformly around the entire circumference, (b) one-half the impulsive force P_I, applied symmetrically about an angle $\theta = 0$ and acting inward on one-half of the wall and one-half P_I applied symmetrical about $\theta = \pi$ and acting outward on the opposite half of the wall (c) one-half the convective force P_C applied symmetrically about an angle

*Modification 1 per Section 3.7.2, Chapter 3

$\theta = 0$ and acting inward on one-half of the wall and one-half P_C applied symmetrical about $\theta = \pi$ and acting outward on the opposite half of the wall; and (d) the dynamic earth and ground water pressure against the trailing half of the buried portion of the tank.

Wall inertia,

$$P_W = \frac{S_{DS}I}{R}W_W = C_{SI}W_W = 0.37 \times 1{,}609 = 595 \text{ kips}$$

Roof Inertia,

$$P_R = \frac{S_{DS}I}{R}W_R = C_{SI}W_R = 0.37 \times 1{,}092 = 404 \text{ kips}$$

Impulsive,

$$P_I = \frac{S_{DS}I}{R}W_I = C_{SI}W_I = 0.37 \times 3{,}763 = 1392 \text{ kips}$$

Convective,

$$P_C = \frac{S_{DS}I}{R}W_C = C_{SC}W_C = 0.034 \times 6{,}988 = 238 \text{ kips}$$

P_{Wy}, P_{Iy} and P_{Cy} and the pressure distribution on the wall are calculated per Section 5.2.1 as follows (see Fig. 8-3)

For wall inertia,

$$P_{Wy} = \frac{P_W}{2H_W} = \frac{595}{2 \times 28} = 10.6 \text{ k/ft}$$

For impulsive force,

$$P_{Iy} = P_I \frac{[4H_L - 6h_I] - \left[(6H_L - 12h_I) \times {}^{y}\!/_{H_L}\right]}{2H_L^2}$$

At bottom of wall,

$$P_{Iy=0} = P_I \frac{[4 \times 26 - 6 \times 9.75] - \left[(6 \times 26 - 12 \times 9.75) \times {}^{0}\!/_{26}\right]}{2 \times 26^2} = 0.034P_I$$

At top liquid level,

$$P_{Iy=26} = P_I \frac{[4 \times 26 - 6 \times 9.75] - \left[(6 \times 26 - 12 \times 9.75) \times {}^{26}\!/_{26}\right]}{2 \times 26^2} = 0.0048P_I$$

For convective force,

$$P_{Cy} = P_C \frac{[4H_L - 6h_C] - \left[(6H_L - 12h_C) \times {}^{y}\!/_{H_L}\right]}{2H_L^2}$$

At bottom of wall,

$$P_{Cy=0} = P_C \frac{[4 \times 26 - 6 \times 14] - \left[(6 \times 26 - 12 \times 14) \times {}^{0}\!/_{26}\right]}{2 \times 26^2} = 0.0148P_C$$

At top liquid level,

$$P_{Cy=26} = P_C \frac{[4 \times 26 - 6 \times 14] - \left[(6 \times 26 - 12 \times 14) \times {}^{26}\!/_{26}\right]}{2 \times 26^2} = 0.0237P_C$$

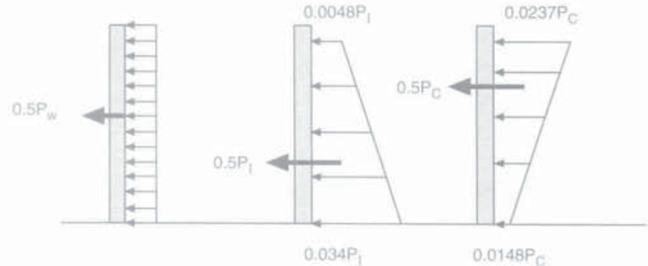


Figure 8-3 Forces on wall perpendicular to the direction of analysis

$$\text{Pressure on wall due to inertia} = \frac{P_{Wy}}{\pi r} = \frac{10.6 \times 1000}{3.14 \times 45} = 75 \text{ psf}$$

(uniform)

Pressure due to impulsive force,

$$\text{Bottom of wall} = \frac{2P_{Iy}}{\pi r}\cos\theta = \frac{2 \times 0.034P_I}{\pi r}\cos\theta$$

$$= \frac{2 \times 0.034 \times 1{,}392 \times 1{,}000}{3.14 \times 45} = 670 \text{ psf}$$

(maximum @ $\theta = 0$)

$$\text{Top of liquid} = \frac{2 \times 0.0048P_I}{\pi r}\cos\theta$$

$$= \frac{2 \times 0.0048 \times 1{,}392 \times 1{,}000}{3.14 \times 45} = 80.7 \text{ psf}$$

(maximum @ $\theta = 0$)

Pressure due to convective force,

$$\text{Bottom of wall} = \frac{16P_{Cy}}{9\pi r}\cos\theta = \frac{16 \times 0.0148P_C}{9\pi r}\cos\theta$$

$$= \frac{16 \times 0.0148 \times 238 \times 1{,}000}{9 \times 3.14 \times 45} = 44.2 \text{ psf}$$

(maximum @ $\theta = 0$)

$$\text{Top of liquid} = \frac{16 \times 0.0237P_C}{9 \times \pi r}\cos\theta = 70.8 \text{ psf}$$

(maximum @ $\theta = 0$)

The above pressure needs to be determined at different values of θ and applied to the leading and the trailing walls to determine the additional hoop stresses and out-of-plane moments. These stresses and moments can be more accurately computed using shell analysis.

Note that currently no design aids are available to determine the hoop stresses and out-of-plane moments due to the above loads. For shallow tanks ($D >> H$) the out-of-plane bending effects are small and can be neglected.

8.4.2.1 Approximate Method

Using ACI 350.3 approximation for tank with $\frac{D}{H} = 4$, 20 % of base shear can be assumed to be transferred through out-of-plane bending.

Approximate force acting on leading and trailing walls $= \frac{0.2 \times 2,405}{2} = 240$ kips

Assuming this force will act at an approximate resultant height of 10 ft (weighted average of h_I and h_C), total approximate out-of-plane maximum moment assuming simply supported wall $= \frac{240 \times 10 \times 18}{28} = 1,543$ ft-kips

Moment per unit length M_E

$= \frac{2 \times 1,543}{3.14 \times 45.67} = 21.5$ ft-kips/ft.

$M_u = 1.3 \times 10.4 + 21.5 = 35$ ft-kips/ft > 23 ft-kips under static load combinations (Ref. 5-2).

No. 6 @ 8 in. ($A_s = 0.66$ in.^{2}) vertical reinforcement on inside face is adequate.

The user should check other loading conditions given in Ref. 5-2 in combination with earthquake loading to verify the adequacy of external vertical reinforcement.

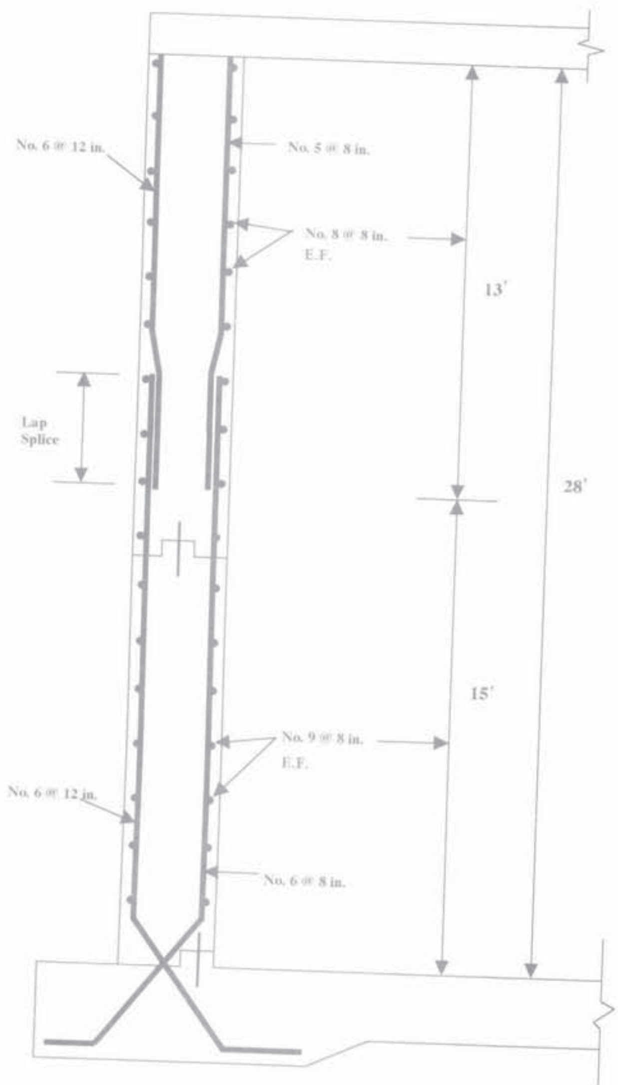


Figure 8-4a Detailing of wall

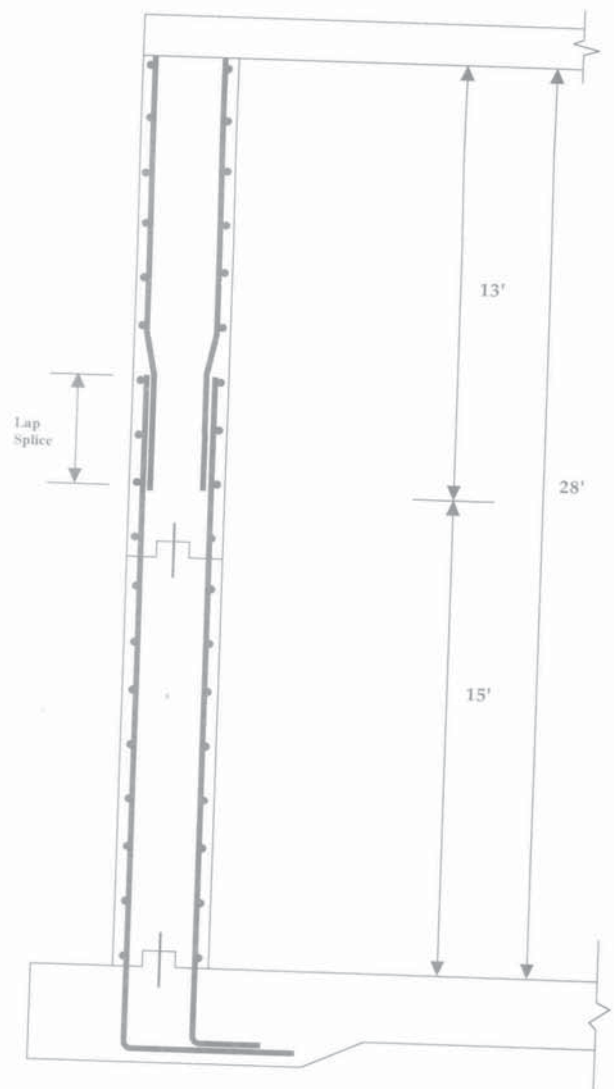


Figure 8-4b Detailing of wall with fixed base

Note that dynamic earth pressure effects are not included in this example. Reference 4-5 should be consulted when determining these effects.

8.5 DETAILING

Based on the SDC *D* associated with this structure, detailing corresponding to high seismic risk will apply per Table 6-1 (see Chapter 6). Based on this, it is prudent to locate the splices away from the potential plastic hinge zones. The Class *B* splice length should be provided for both No. 6 vertical bars as well as No. 8 and No. 9 horizontal bars per 12.2 of ACI 318-99. The required development length of the No. 6 and No. 7 dowels in the base slab is 18 in.

The requirements of Table 6-3 are satisfied for in-plane wall design. The steel provided along with spacing satisfies Section 21.6.2.1 of ACI 318 (Table 6-3). The reinforcement provided also satisfies minimum requirements of ACI 350 (ρ_{min} = 0.003 and maximum spacing = 12 in.). Per Section 21.6.2.3 of ACI 318-99, all continuous reinforcement in structural walls should be anchored or spliced in accordance with the provisions of reinforcement in tension (21.5.4). Figure 8-4a shows the detailing of the wall with hinged base.

A tank with fixed base detail as shown in Figure 8-4b may be more appropriate for ease of construction and also to limit crack opening at the base of wall. The designer will have to analyze the tank and determine the appropriate reinforcement required for this condition.

Notation

A_a = effective peak acceleration (BOCA 1996, SBC 1997)
A_s = cross-sectional area of base cable, strand, or conventional reinforcement, in^2 (mm^2)
A_v = effective peak velocity-related acceleration (BOCA 1996, SBC 1997)
b = one-half of projected width or radius of immersed element, in. (mm)
B = inside length of a rectangular tank perpendicular to the direction of the earthquake force, ft (m)
C_S = seismic response coefficient per IBC 2000 (C_{SI} = impulsive coefficient, C_{SC} = convective coefficient)
C_a = acceleration dependent seismic coefficient (UBC 1997)
C_v = velocity dependent seismic coefficient (UBC 1997)
C_W = coefficient for determining the fundamental frequency of circular tank (see Fig. 4-10)
d_{max} = freeboard (sloshing height) measured from the liquid surface at rest, ft (m)
D = inside diameter of circular tank, ft (m)
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 = site coefficient based on IBC 2000
F_v = site coefficient based on IBC 2000
G = shear modulus, lb/in.^{2} (MPa)
G_p = shear modulus of elastomeric bearing pad, lb/in.^{2} (MPa)
g = acceleration due to gravity (32.17 ft/sec^2, 9807 mm/sec^2)
h = mean height at which the inertia force of tank and its contents is assumed to act, ft (m)
h_C = height from base of wall to the center of the convective force, ft (m)
h_I = height from base of wall to the center of impulsive force, ft (m)
h_R = height from base of the wall to the center of gravity of the tank roof, ft (m)
h_W = height from the base of wall to the center of inertia of the tank shell, ft (m)
H_L = design depth of stored liquid, ft (m)
H_W = wall height (inside dimension), ft (m)
I, I_p = Importance Factor (IBC 2000, UBC 1997, UBC 1994, BOCA 1996, SBC 1997, ACI 350.3)
k_a = spring constant of the tank wall support system, lb/ft^2 (kPa)
k_o = coefficient of lateral earth pressure at rest
K = flexural stiffness of tank wall, lb/ft^2 (kPa)
L = inside length of a rectangular tank parallel to the direction of the earthquake force, ft (m)
L_p = length of individual elastomeric bearing pads, in. (mm)
L_s = effective length of cable or strand taken as the sleeve length plus 35 times the strand diameter, in. (mm)
M_C = overturning moment due to convective force, ft-lb (kN-m)
M_I = overturning moment due to impulsive force, ft-lb (kN-m)
M_T = total overturning moment based on SRSS combination of impulsive and convective moments, ft-lb (kN-m)
M_n = nominal moment strength, ft-lb (kN-m)
M_u = required moment strength, ft-lb (kN-m)
N_a, N_v = near source factors (UBC 1997)
N_{Cy} = hoop force in circular tanks at level y due to the convective component, pounds per foot of wall height, lb/ft (kN/m)
N_{Iy} = hoop force in circular tanks at level y due to the impulsive component, pounds per foot of wall height, lb/ft (kN/m)
N_{Wy} = inertia force in circular tanks at level y, pounds per foot of wall height, lb/ft (kN/m)
N_y = total effective hoop force in circular tanks at level y, pounds per foot of wall height, lb/ft (kN/m)
p_{hy} = hydrostatic pressure at level y above the base of the tank, lb/ft^2 (kPa)
p_{cy} = unit lateral dynamic convective pressure distributed horizontally at level y, lb/ft^2 (kPa)
p_{iy} = unit lateral dynamic impulsive pressure distributed horizontally at level y, lb/ft^2 (kPa)

P_{wy} = unit lateral inertia force due to wall dead weight distributed horizontally at level y, 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 level y, pounds per ft of wall height, lb/ft (kN/m)

P_E = dynamic earth pressure, lb (kN)

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 at liquid level y, pounds per ft of wall height, lb/ft (kN/m)

P_R = inertia force of the accelerating roof of weight W_R, lb (kN)

P_W = lateral inertia force of the accelerating wall of weight W_W, lb (kN)

P_{Wy} = lateral inertia force per unit height of the tank wall occurring at level y, pounds per ft of wall height, lb (kN/m)

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

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

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

Q_E = effect of horizontal seismic force

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

R, R_W = response modification factor, (R_{wc} for the convective component of the accelerating liquid; R_{wi} for the impulsive component)

S = soil factor or soil profile type (IBC 2000, UBC 1997, UBC 1994, BOCA 1996, SBC 1997, ACI 350.3)

S_a = generalized design spectral response acceleration corresponding to a given natural period, T, (S_{aI} corresponds to T_I and S_{aC} corresponds to T_C)

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

S_{DS} = design spectral response acceleration at short period per IBC 2000

S_{D1} = design spectral response acceleration at 1 second per IBC 2000

S_{MS} = maximum considered earthquake spectral response acceleration at short period per IBC 2000

S_{M1} = maximum considered earthquake spectral response acceleration at 1 second per IBC 2000

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

S_S = the mapped spectral accelerations for short period per IBC 2000

S_1 = the mapped spectral accelerations for 1 second period per IBC 2000

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

t_w = average wall thickness, in. (mm)

T = fundamental period, sec.

T_0 = $0.2T_s = 0.2S_{D1}/S_{DS}$

T_C = natural period of the first (convective) mode of sloshing, sec.

T_I = fundamental period of tank and its contents, sec.

T_S = S_{D1}/S_{DS}

T_V = natural period of vibration of vertical liquid motion, sec.

$\ddot{u}_v$ = magnitude of vertical acceleration associated with T_V, ft/sec^2 (m/sec^2)

V = total horizontal base shear, lb (kN)

V_C = convective base shear, lb (kN)

V_I = impulsive base shear, lb (kN)

V_T = total base shear based on SRSS combination of impulsive and convective base shear, lb (kN)

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

W = effective seismic weight of the structure, lb (kN)

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

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

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

W_P = weight of tank and contained liquid, lb (kN)

W_R = weight of roof, lb (kN)

W_T = total weight, lb (kN)

W_W = weight of the tank walls (shell), lb (kN)

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

Z = zone factor (UBC 1997, UBC 1994)

α = added weight ratio for immersed elements

β = angle of base cable or strand with horizontal, deg

γ_c = unit mass of concrete (150 lb/ft^3, 23.56 kN/m^3 for normal-weight concrete)

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

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

ϕ = strength reduction factor (ACI 318)

λ = coefficient for determining T_C for circular tanks (see Fig. 4-9)

ρ_c = mass density of concrete (4.66 lb-sec^2/ft^4, 2.40 kN-sec^2/m^4 for normal-weight concrete)

ρ_L = mass density of the contained liquid ($\gamma_L = g_L/g$), lb-sec^2/ft^4 (kN-sec^2/m^4)

ρ_w = mass density of water (1.94 lb-sec2/ft$_4$, 1.0 kN-sec^2/m^4)

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

ω_I = circular frequency of the impulsive mode, rad/sec

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