

ACI 371R-16

Guide for the Analysis, Design, and Construction of Elevated Concrete and Composite Steel- Concrete Water Storage Tanks

Reported by ACI Committee 371



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Guide for the Analysis, Design, and Construction of Elevated Concrete and Composite Steel-Concrete Water Storage Tanks

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This guide presents recommendations for materials, analysis, design, and construction of concrete-pedestal elevated water storage tanks, including all-concrete and composite tanks. Composite tanks consist of a steel water storage vessel supported on a cylindrical reinforced concrete pedestal.

Concrete-pedestal elevated water storage tanks are structures that present special problems not encountered in typical environmental engineering concrete structures. This guide refers to ACI 350 for design and construction of those components of the pedestal tank in contact with the stored water, and to ACI 318 for design and construction of components not in contact with the stored water. Determination of snow, wind, and seismic loads based on ASCE/SEI 7 is included. These loads conform to the requirements of national building codes that use ASCE/SEI 7 as the basis for environmental loads as well as those of local building codes. Special requirements, based on successful experience, for the unique aspects of loads, analysis, design, and construction of concrete-pedestal tanks are presented.

Keywords: composite tanks; concrete-pedestal tanks; earthquake-resistant structures; elevated water tanks; formwork (construction).

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ACI 371R-16 supersedes ACI 371R-08 and was adopted and published June 2016.
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CHAPTER 1—GENERAL**1.1—Introduction**

This guide provides recommendations for the design and construction of elevated concrete and composite steel-concrete water storage tanks based on practices used in successful projects. Elevated tanks are used by municipalities and industry for potable water supply and fire protection. Commonly built sizes of elevated concrete and composite steel-concrete water storage tanks range from 500,000 to 3,000,000 gal. (1900 to 11,000 m³). Concrete pedestal heights range from 25 to 200 ft (8 to 60 m), depending on water system requirements and site elevation. The interior of the concrete pedestal may be used for material and equipment storage, office space, and other applications.

Since the 1970s, concrete-pedestal elevated water storage tanks have been constructed in North America with a steel water-containing element and an all-concrete support structure. The generic term “composite elevated tank” is often used to describe tanks of this configuration. A few

all-concrete elevated tanks have been built in the United States throughout the last century, as well as a few elevated prestressed tanks jacked into place. Elevated post-tensioned tanks as detailed in this guide have a long history in Europe, and were introduced to the U.S. market in the 1990s.

All-concrete and composite steel concrete elevated tanks are competitively marketed as complete entities, including design, and are constructed under design-build contracts using proprietary designs, details, and methods of construction. The designs, however, are frequently reviewed by owners and their consulting engineers, or by city or county officials.

Elevated tanks designed and constructed in accordance with the recommendations of this guide are expected to be durable structures that require only routine maintenance. Details of concrete surfaces that promote good drainage and avoid low areas conducive to ponding essentially eliminate the problems associated with cyclic freezing and thawing of fresh concrete in cold climates. The quality of concrete for elevated tanks in this guide meets the requirements for durable concrete as defined in **ACI 201.2R**. It has adequate strength, a low water-cementitious materials ratio (*w/cm*), and air entrainment for frost exposure. The concrete support structure loads are primarily compressive with little or no cyclic loading with stress reversal.

1.2—Scope

Recommendations supplement the general requirements for reinforced concrete and prestressed concrete design and construction given in **ACI 318**, **ACI 301**, **ACI 350**, and **ACI 350.5**. Design and construction recommendations include materials, determination of structural loads, design of concrete elements including foundations, design of concrete or steel tank components, construction requirements, geotechnical requirements, appurtenances, and accessories. Materials, design, fabrication, and construction of the steel vessel of composite steel-concrete tanks are addressed by applicable sections of **AWWA D100**.

Design and construction recommendations are presented for the types of elevated concrete and composite steel-concrete water storage tanks shown in Fig. 1.2a and 1.2b. The elevated concrete tank consists of a post-tensioned concrete vessel on a cast-in-place concrete pedestal. The composite steel-concrete tank consists of a steel vessel on a cast-in-place concrete pedestal.

This guide may be used in whole or part for other tank configurations; however, the designer should determine the suitability of such use for other configurations and details.

1.3—Construction documents

Construction documents should show all features of the work, including:

- (a) Tank capacity
- (b) Codes and standards used in design
- (c) Design basis and loads used in design
- (d) Size and position of structural components and reinforcement
- (e) Structural details
- (f) Specified concrete compressive strength

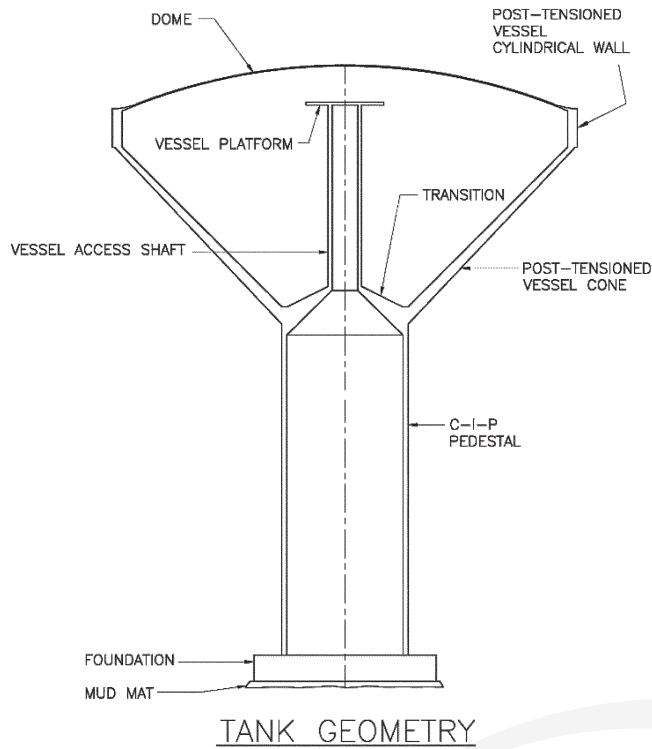


Fig. 1.2a—Common configuration of elevated concrete tanks.



Fig. 1.4a—Completed composite elevated tank (photo courtesy of Landmark).

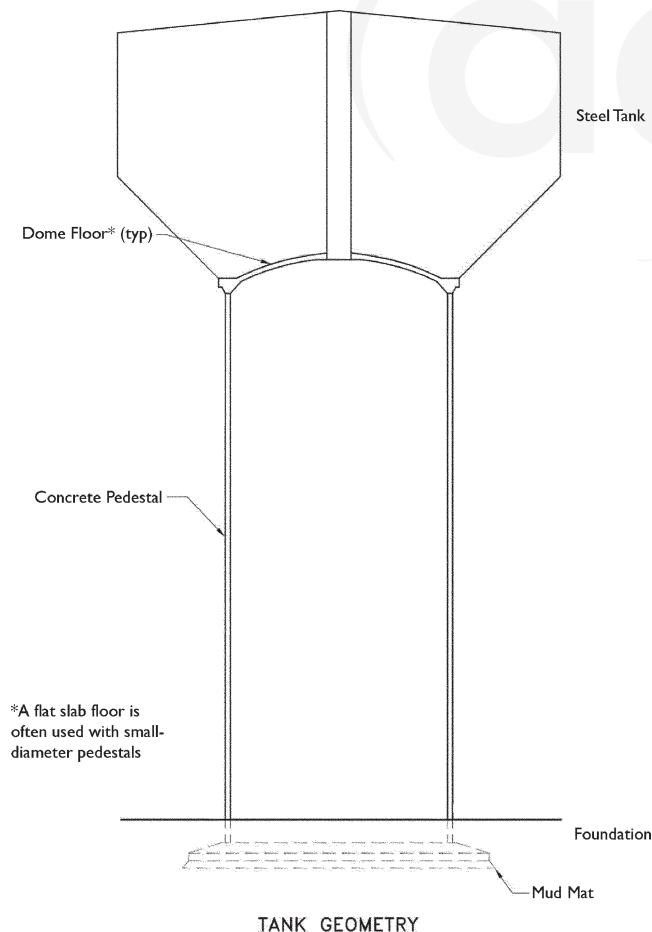


Fig. 1.2b—Common configuration of elevated composite steel-concrete tanks.

(g) Strength or grade of reinforcement and structural steel
Where the tank builder is providing both design and construction of the tank, full calculations detailing the structural aspects of the tank should be provided to the owner or owner's agent.

1.4—Sample tank photos

This section presents photographs of numerous varieties of tanks (Fig. 1.4a to 1.4h).

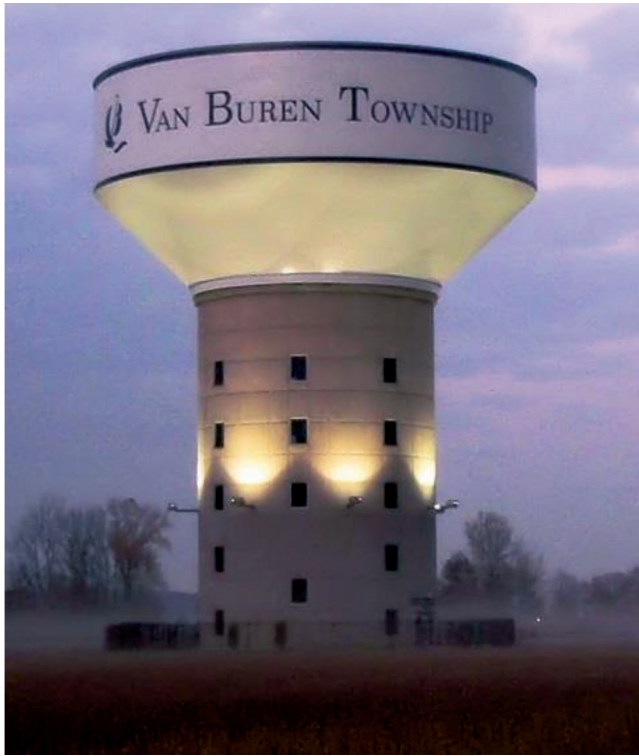


Fig. 1.4b—Completed composite elevated tank (photo courtesy of CBI).



Fig. 1.4d—Completed concrete elevated tank (photo courtesy of Crom LLC).



Fig. 1.4c—Completed composite elevated tanks (photo courtesy of Caldwell Tanks, Inc.).



Fig. 1.4e—Construction of composite elevated tank (photo courtesy of Landmark).



Fig. 1.4f—Construction of composite elevated tank pedestal (photo courtesy of CBI).



Fig. 1.4g—Construction of composite elevated tank pedestal (photo courtesy of Caldwell Tanks, Inc.).



Fig. 1.4h—Construction of concrete elevated tank (photo courtesy of Crom LLC).

CHAPTER 2—NOTATION AND DEFINITIONS

2.1—Notation

A	=	area
A_c	=	area of gross section of pedestal
A_{cv}	=	concrete shear area of a section, in. ² (mm ²)
A_f	=	horizontal projected area of a portion of the structure where the wind force coefficient C_f and the wind pressure p_z are constant, in. ² (mm ²)
A_g	=	gross concrete area of a section, in. ² (mm ²)
A_s	=	area of nonprestressed tension reinforcement, in. ² (mm ²)
A_t	=	cross-sectional area of vessel at mid-depth of water
A_w	=	gross horizontal cross-sectional concrete area of wall, in. ² (mm ²), per unit length of circumference, ft (m)
b_d	=	width of a doorway or other opening, in. (mm)
b_v	=	equivalent shear wall length not to exceed $0.78d_w$, in. (mm)
b_x	=	cumulative opening width in a distance of b_v , in. (mm)
C	=	buckling parameter for thin metal cylinders that buckle with diamond-shaped pattern
C_c	=	spectral acceleration of sloshing liquid
C_e	=	eccentricity coefficient that accounts for the resultant of factored axial load being eccentric to the centroid of the pedestal thickness
C_{es}	=	snow load exposure factor
C_f	=	wind force coefficient
C_s	=	seismic response coefficient
C_{sm}	=	modal seismic design coefficient for mode m
C_{vx}	=	seismic distribution factor
C_{vxm}	=	seismic distribution factor of the m -th mode
c_c	=	clear cover from the nearest surface in tension to the surface of the flexural torsion reinforcement, in. (mm)
D	=	dead load
D_t	=	tank diameter at the water surface, ft (m)
d_c	=	distance from the closest face to the centroid of the tension reinforcement, in. (mm)
d_w	=	mean diameter of concrete pedestal, ft (m)
E	=	combined effect of horizontal and vertical earthquake forces
E_c	=	modulus of elasticity for concrete, psi (MPa)
E_{cr}	=	modulus of elasticity for concrete to allow for microcracking and creep, psi (MPa)
e	=	eccentricity of the axial wall load, in. (mm)
e_g	=	vertical load eccentricity, in. (mm)
e_o	=	minimum vertical load eccentricity, in. (mm)
F	=	weight and pressure of stored water
F_a	=	seismic acceleration-based site coefficient
F_i	=	portion of the total seismic shear V acting at level i , kip (kN)
F_s	=	factor of safety against buckling
F_v	=	seismic-velocity-based site coefficient
F_x	=	portion of the seismic shear V acting at level x , kip (kN)
F_{xm}	=	modal force at each level, kip (kN)
F_z	=	wind force acting on tributary area A_f , kip (kN)
f_c	=	concrete compressive stress
f_{ca}	=	average compression stress
f'_c	=	specified compressive strength of concrete, psi (MPa)

$\sqrt[3]{f'_c}$ = square root of specified compressive strength, psi (MPa)	p_r = rain on snow surcharge, lb/ft ² (N/m ²)
f_s = calculated stress in reinforcement at service loads for control of cracking, ksi (MPa)	p_s = snow load on roof, lb/ft ² (N/m ²)
f_y = specified yield strength of reinforcing steel, psi (MPa)	p_z = wind pressure at height z , lb/ft ² (N/m ²)
G = eccentric load effects due to dead load and water	Q_a = allowable service load capacity of a pile or pier, kip (kN)
G_w = wind gust factor	Q_r = ultimate capacity of a pile or pier, kip (kN)
g = acceleration due to gravity, 32.2 ft/s ² (9.8 m/s ²)	q_a = allowable bearing capacity of a shallow foundation, lb/ft ² (N/m ²)
h = height from grade to the top of the vessel shell, ft (m)	q_r = ultimate bearing capacity of a shallow foundation, lb/ft ² (N/m ²)
h_d = height of a doorway opening, ft (m)	q_z = velocity pressure evaluated at height z above ground, lb/ft ² (N/m ²)
h_f = foundation depth measured from original ground line, ft (m)	R = response modification coefficient (for seismic) or rain load (for roof load)
h_i = height from the base to level i for seismic force distribution, ft (m)	R_d = mean meridional radius of dome tank floor, ft (m)
h_r = wall thickness exclusive of any rustications or architectural relief, in. (mm)	S = snow load
h_s = steel vessel dome tank floor thickness, in. (mm)	S_1 = maximum considered earthquake spectral response acceleration at 1 second
h_x = height from the base to level x for seismic force distribution, ft (m)	S_{D1} = seismic design spectral response acceleration for a period of 1 second
I = importance factor	S_{DS} = seismic design spectral response acceleration for short period
I_c = moment of inertia of the gross concrete, section of pedestal	S_s = maximum considered earthquake spectral response acceleration at short periods
I_t = moment of inertia of vessel at mid-depth of water	s = spacing of nonprestressed tension reinforcement, in. (mm)
K_z = wind exposure coefficient	T = force due to restrained thermal movement, creep, shrinkage, or differential settlement
k = structure exponent	T_f = fundamental period of vibration of structure, seconds
k_c = structural lateral stiffness used for composite tank period calculation	T_L = lower bound of the period at which the long period response spectrum starts, seconds
k_ℓ = effective unsupported column length, ft (m)	T_m = modal period of vibration of the m -th mode, seconds
k_s = effective length factor	T_s = S_{D1}/S_{DS}
k_w = crack width constant	V = total design lateral force or shear at base of structure, kip (kN)
L = interior floor live loads	V_b = basic wind speed, mph (m/s)
L_g = actual height from base to centroid of stored water	V_m = portion of the seismic base shear contributed by the m -th mode, kip (kN)
L_{ga} = approximate height from base to centroid of stored water	V_n = nominal shear strength, kip (kN)
L_p = pedestal length above base slab	V_u = factored shear force, kip (kN)
L_r = roof live load	V_x = lateral seismic shear acting at any level of the structure, kip (kN)
L_t = length of vessel from top of pedestal to midheight of water depth	W = wind load effect
ℓ_{cg} = length of pedestal acting as a cantilever beam	W_c = weight of concrete below grade, lb (N)
ℓ_g = distance from bottom of foundation to centroid of stored water, in. (mm)	W_e = seismic effective weight, kip (kN)
ℓ_w = length of a segment considered in direction of shear force	W_I = weight of water and structure above base slab, lb (N)
M_h = wind ovalling moment per unit of height at horizontal sections, kip-ft/ft (kN-m/m)	W_i = portion of the total weight whose centroid is at level i , lb (N)
M_u = factored moment, kip-ft/ft (kN-m/m)	W_m = effective modal weight
M_{uw} = factored axial wall moment per unit of circumference, ft-lb/ft (N-m/m)	W_r = horizontal distance from eaves to center of cone, conical, or curved roof, ft (m)
M_x = seismic overturning moment at height h_x above base, kip-ft (kN-m)	W_s = weight of soil below grade, lb/ft ² (N/m ²)
P = foundation load above grade, kip (kN)	W_x = portion of the total weight whose centroid is at level x , lb (N)
P_{cr} = critical buckling load	w = calculated crack width, in. (mm)
P_{mw} = nominal axial load strength of wall per unit of circumference, lb/ft (N/m)	w_u = factored distributed load, lb/ft ² (N/m ²)
P_s = gravity service load, kip (kN)	Z = parameter used to delineate buckling regions
P_{uw} = factored axial wall load, lb (N) per unit of circumference, ft (m)	
p_g = ground snow load, lb/ft ² (N/m ²)	

z	=	height above ground level, ft (m)
z_g	=	nominal height of the atmospheric boundary layer
z_{15}	=	15 ft (4.6 m) in height
α	=	three-second gust-speed power law exponent
α_c	=	constant used to compute in-plane nominal shear strength
α_f	=	geometric term in all-concrete tank fundamental period of vibration of structure
β	=	ratio of distances to the neutral axis from the extreme tension fiber and from the centroid of the main reinforcement
δ_i	=	static elastic deflection of the structure at level i due to forces F_i
δ_s	=	height of sloshing wave, ft (m)
Φ	=	modal shape factor used for seismic analysis
Φ_{im}	=	displacement amplitude at the level i of the structure when vibrating in its m -th mode
Φ_{xm}	=	displacement amplitude at the level x of the structure when vibrating in its m -th mode
ϕ	=	strength reduction factor
γ_s	=	unit weight of soil, lb/ft ³ (N/m ³)
θ	=	half-angle of effective shear wall perimeter for pedestal, degree
θ_g	=	foundation tilting angle
θ_r	=	slope of a straight line from the eaves, or the 70-degree point if present, to the crown of a curved, conical, or cone roof, degrees
ρ	=	A_s/bd , ratio of nonprestressed tension reinforcement
ρ_g	=	A_s/A_g , ratio of total nonprestressed reinforcement
ρ_h	=	ratio of horizontal distributed shear reinforcement on a horizontal plane
ρ_v	=	ratio of vertical distributed shear reinforcement on a vertical plane perpendicular to A_{cv}
ψ	=	wall opening ratio

2.2—Definitions

ACI provides a comprehensive list of definitions through an online resource, “ACI Concrete Terminology,” <https://www.concrete.org/store/productdetail.aspx?ItemID=CT13>. Definitions provided herein complement that resource.

cone roof—roof with a constant slope from the crown to the eaves.

conical roof—cone roof combined with a steeply sloped edge cone or a doubly curved edge segment.

crown—highest point of the roof at centerline of tank.

curved roof—roof with a continuously increasing slope from crown to eaves; or the doubly curved portion of a spherical roof.

deep foundation—foundation with a bottom elevation typically far below the soil surface that obtains its load-carrying capacity through a combination of direct bearing on soil or rock at its base and friction or adhesion on its vertical sides.

eave—highest level at which the tank diameter is at a maximum or the roof slope of curved or conical roof is 70 degrees from horizontal.

ringbeam—continuous tension and flexural element tied to the top or bottom perimeter of a cylindrical structure.

roof slope θ_r —slope of a straight line from the eaves (or the 70-degree point if present) to the crown of a curved roof, a conical roof, or a cone roof.

shallow foundation—foundation with a bottom elevation typically near the soil surface that obtains its load-carrying capacity primarily by direct bearing on soil or rock rather than friction or adhesion on its vertical sides.

tank vessel—water-containing portion of the structure.

top line capacity—water height to the lip of the tank overflow.

vessel floor—structural concrete cone, dome, or flat slab that supports the vessel contents inside the pedestal.

CHAPTER 3—MATERIALS

3.1—Materials common to both composite and concrete tank types

3.1.1 General—Concrete that is not in contact with stored water should meet the requirements of **ACI 318** except as indicated in 3.1.2 through 3.1.4.

3.1.2 Compressive strength—The minimum specified compressive strength of concrete should be:

- Pedestal: 4000 psi (28 MPa)
- Foundations and intermediate floors: 3500 psi (24 MPa)
- Slabs-on-ground (Table 3.1.2)

3.1.3 Admixtures—Admixtures may be used to achieve the required properties of concrete, provided that the correct type and proper dosage are used. Admixtures should be compatible such that their combined effects produce the required results in hardened concrete, as well as during placement and curing.

3.1.3.1 Chemical admixtures—Chemical admixtures should meet the requirements of **ASTM C494/C494M** and be used in accordance with **ACI 350.5**. Air-entraining admixtures should comply with **ASTM C260/C260M**. High-range water-reducing admixtures conforming to ASTM C494/C494M Type F or G can be used to facilitate the placement of concrete. Shrinkage-reducing admixtures can be used to reduce the effects of drying shrinkage in concrete. These admixtures should meet the requirements of ASTM C494/C494M Type S.

Table 3.1.2—Minimum recommendations for slabs-on-ground

Anticipated loadings	Foot traffic	Light vehicular traffic (<15 ton axle load)	Heavy vehicular traffic (> 15 ton axle load)
Minimum recommended slab thickness	4 in. (100 mm)	5 in. (125 mm)	6 in. (150 mm)
Minimum recommended concrete strength f'_c	3500 psi (24 MPa)		4000 psi (28 MPa)
Minimum recommended reinforcement ratio	0.0018		

Notes:

1. The design of slabs-on-ground should be in accordance with the recommendations provided in ACI 360R and the Post-Tensioning Institute's (PTI) DC10.1.

2. Although slabs-on-ground are not covered by ACI 318, the design and construction requirements in this table are based on the shrinkage and temperature steel requirements in accordance with ACI 318 and have performed well in full-scale tests.

3.1.4 Supplementary cementitious materials—Supplementary cementitious materials such as fly ash, blast-furnace slag, silica fume, and natural pozzolanic material can be used in concrete. When added in the proper amounts, these materials have been successful in reducing the effects of drying shrinkage, mitigating the effects of alkali-silica reactivity, reducing heat of hydration, and reducing permeability. The benefits of supplementary cementitious materials are more fully discussed in [ACI 318](#) and [ACI 232.2R](#). Maximum limits on supplementary cementitious materials are critical to achieve concrete mixtures that demonstrate proper strength and acceptable durability. Supplementary materials should be added in accordance with the requirements of [ACI 318](#) and the recommendations provided in [ACI 301](#).

3.2—Materials specific to composite tanks

Materials used for the steel vessel of composite water tanks should comply with recognized standards such as [AWWA D107](#) and applicable sections of [AWWA D100](#).

3.3—Materials specific to concrete tanks

3.3.1 General—Materials for concrete tank components in contact with stored water should meet the requirements of ACI 350, as supplemented by these recommendations. Prestressed reinforcement, materials related to prestressing, waterstops, sealing materials, and coatings for outer surfaces for concrete tanks should be in accordance with [ACI 350](#), except as indicated.

3.3.2 Specified compressive strength—The minimum specified compressive strength of concrete should conform to:

- (a) Tank vessel prestressed elements: 5000 psi (35 MPa)
- (b) Tank vessel floor, dome, access shaft and platform: 4000 psi (28 MPa)

3.3.3 Injectable sealant tubes—Injectable sealant tubes may be used as a delivery system for waterproofing resins that are injected into the structure after construction is complete. These tubes may be used in addition to a PVC waterstop at construction joints as a preventative measure against leakage. Injectable sealant tubes should be completely permeable from the interior of the tube to allow injected sealing materials to exit the tube while also being impermeable against the intrusion of cementitious materials used in construction.

3.3.4 Polyurethane grout—Injectable waterstop tubes should be injected with a polyurethane grout after construction is complete to fill imperfections in the concrete construction joint and prevent leakage. Polyurethane grout should be a one- or two-component, 100-percent-solids material meeting the requirements of [ASTM D2369](#) and [ASTM C1583/C1583M](#). Polyurethane grout should exhibit expansive properties when exposed to moisture and have adequate flexibility for the application.

3.3.5 Epoxy adhesives—The bond between hardened concrete and fresh concrete may be increased by proper use and application of an epoxy adhesive. The epoxy adhesive should be a two-component, 100-percent-solids, moisture-insensitive epoxy adhesive meeting [ASTM C881/C881M](#)

Type II, Grade 2. [ACI 503.2](#) also contains information on this subject. The bonding agent should produce a bond strength ([ASTM C882/C882M](#)) not less than 1500 psi (10.3 MPa) 14 days after the fresh concrete is placed. Refer to [ACI 515.2R](#) for further information on epoxy adhesives and their use in concrete construction, as well as recommendations on surface preparation before application of the adhesives.

3.3.6 Shotcrete—Shotcrete may be used in elevated concrete tank construction and should be used as recommended in [ACI 372R](#) and [506R](#).

CHAPTER 4—DESIGN

4.1—General recommendations common to both composite and concrete tank types

4.1.1 Scope—This chapter recommends design and analysis procedures for elevated concrete and composite steel-concrete storage water tanks.

4.1.2 Design recommendations for foundation—The foundation should be designed to provide sufficient strength to support the tank vessel and pedestal. The foundation and its connection to the pedestal should be designed to resist axial loads, shears, and moments determined from an analysis of the loadings in 4.2.2. The foundation should provide capacity to resist the seismic forces defined in 4.2.8 against sliding, overturning, and uplift, and should be designed with the details in 4.3.4 that promote ductility and energy dissipation. The methods of allowable stress design may be used to determine the stresses at the foundation/earth interface due to bearing, sliding, and overturning. The foundation should extend over a sufficient area to preclude foundation uplift caused by wind or seismic overturning moment.

4.1.3 Design recommendations for pedestal—The pedestal should be designed to provide sufficient strength to support the tank vessel. The pedestal and its connection to the tank vessel should be designed to resist the axial forces, shears, and moments determined from an analysis of the loadings in 4.2.2. The pedestal should provide capacity to resist the seismic forces defined in 4.2.8, and should be designed with the details in 4.3.5 that promote ductility and energy dissipation. Design of the pedestal should conform to [ACI 318](#) and design recommendations of this guide. The pedestal design should also accommodate access to the tank vessel.

4.1.4 Design recommendations for tank vessel—The tank vessel should be designed to provide watertight containment of tank contents within the vessel, as well as protection of the contained water from environmental contamination from without. The tank vessel should be designed to resist the axial forces, shears, and moments determined from analysis based on loading combinations in 4.2.2.

4.1.5 Design recommendations for other concrete members—Design of concrete members such as floor slabs and similar structural members should conform to [ACI 318](#) and the recommendations of 7.8.

4.1.6 Design recommendations for non-concrete members—Design of non-concrete-related elements, such as structural steel framing members, should conform to recognized national standards ([ANSI/AISC 360](#)).

4.1.7 Design recommendations for industrial safety-related components—Handrails, ladders, platforms, and similar safety-related components should conform to the applicable building code, and to U.S. Occupational Safety and Health Administration (OSHA) standards.

4.1.8 Strength recommendations: concrete structure—Concrete portions of the structure should be designed to resist the applied loads that may act on the structure and should conform to this guide.

4.1.8.1 Specified concrete strength—Specified compressive strength f'_c of concrete components should conform to 3.3.2, 4.3.5.2, and 4.4.3.1.

4.1.8.2 Specified strength for reinforcement—The specified yield strength of nonprestressed reinforcement, f_y , should not exceed 60,000 psi (412 MPa).

4.1.9 Design methods

4.1.9.1 Strength design method—Structural concrete members not in contact with the stored water should be proportioned for adequate strength in accordance with the strength design provisions of ACI 318 and this guide. Structural concrete members in contact with the stored water should be proportioned for adequate strength in accordance with strength design provisions of ACI 350. Loads should not be less than the factored loads and forces in 4.2.2.1. Strength reduction factors ϕ should conform to ACI 318-99 and 4.3.8.5, 4.3.10.6, and 4.4.3.2 of this guide.

4.1.9.2 Alternate design method—The alternate design method of ACI 318-99 or ACI 350 for concrete members not in contact with stored water and ACI 350 for concrete members in contact with stored water are acceptable methods for design. Unfactored load combinations should conform to 4.2.2.2.

4.1.10 Minimum reinforcement

4.1.10.1 Flexural members—Where flexural reinforcement is required by analysis, the minimum reinforcement ratio ρ should be at least the greater of $3\sqrt{f'_c}/f_y$ and $200/f_y$ in inch-pound units ($0.25\sqrt{f'_c}/f_y$ and $1.4/f_y$ in SI units). A smaller amount of reinforcement may be used if, at every section, the area of tensile reinforcement provided is at least one-third greater than that required by analysis.

4.1.10.2 Direct tension members—For nonprestressed structural elements in regions of direct tension exceeding 100 psi (0.7 MPa), the minimum reinforcement ratio ρ_g should not be less than $5\sqrt{f'_c}/f_y$ ($0.42\sqrt{f'_c}/f_y$). A smaller amount of reinforcement may be used if the area of tensile reinforcement provided is at least one-third greater than that required by analysis. The minimum reinforcement ratio for regions of significant tension stress is based on equating the cracking strength of plain concrete to f_y . The direct tension cracking strength is taken equal to two-thirds the modulus of rupture, $7.5\sqrt{f'_c}$ in inch-pound units ($(5/8)\sqrt{f'_c}$ in SI units). This requirement is intended to prevent abrupt strength changes when cracking occurs.

4.1.11 Serviceability recommendations: concrete structure

4.1.11.1 General—Concrete portions of the elevated tank should conform to this guide to provide adequate performance at service loads. The following should be considered:

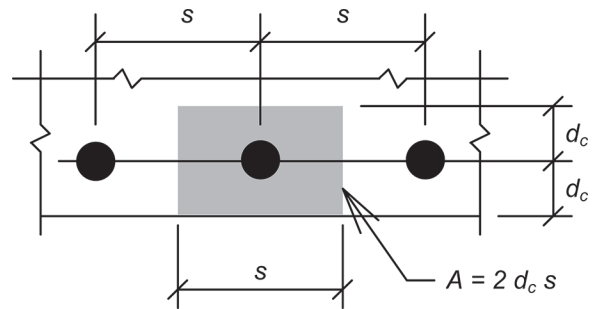


Fig. 4.1.11.2—Effective tension area of concrete.

(a) Deflection of flexural beam or slab elements should conform to the limits of ACI 318 or 350, as applicable.

(b) Control of cracking should conform to 4.1.11.2, 4.4.2.4, and 4.4.3.4.

(c) Settlement of foundations should conform to 6.3 and 6.5.

4.1.11.2 Distribution of flexural and tension reinforcement—Distribution of flexural and tension reinforcement to control cracking should be considered at locations where analysis indicates flexural tension or direct tension stresses occur, and should follow ACI 350.

Calculated stress in reinforcement, f_s , is for Load Combination S1.1 in 4.2.2.2(b). Alternatively, f_s may be taken as 60 percent of the specified yield strength f_y .

Equations (4.1.11.2a) and (4.1.11.2b) provide a distribution of reinforcement that will reasonably control flexural cracking and is also recommended for controlling cracking due to direct tension. It follows the approach of ACI 318 of emphasizing reinforcing details rather than actual crack width calculations.

Equations (4.1.11.2a) and (4.1.11.2b) are not specifically intended for members subject to direct tension; for these members, it is recommended to limit calculated crack width w to 0.013 in. (0.33 mm) using Eq. (4.1.11.2a). Equation (4.1.11.2a) is the Gergely-Lutz expression that was developed for flexural members and was the basis for distribution of flexural reinforcement in prior to the ACI 318-99 edition using the z_c factor. Numerically, z_c is equal to $w/(k_w\beta)$

$$w = k_w \beta f_s \sqrt[3]{d_c A} \quad (4.1.11.2a)$$

where $k_w = 76 \times 10^{-6} \text{ in.}^2/\text{kip}$ ($11 \times 10^{-6} \text{ mm}^2/\text{N}$) for members subject to flexure and $100 \times 10^{-6} \text{ in.}^2/\text{kip}$ ($14.5 \times 10^{-6} \text{ mm}^2/\text{N}$) for members subject to direct tension, and $\beta = 1.2$ for members subject to flexure and 1.0 for members subject to direct tension (Fig. 4.1.11.2).

The direct tension limit is based on the following crack width equation (Eq. (4.1.11.2b)) for direct tension given in ACI 224.2R

$$w = 0.138 f_s d_c \sqrt{1 + \left(\frac{s}{4d_c}\right)^2} \approx 0.10 f_s \sqrt[3]{d_c A} \quad (4.1.11.2b)$$

for s/d_c between 1 and 2.



Documents

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4.2—Load recommendations common to both composite and concrete tank types

4.2.1 General—The elevated water storage tank and its elements and connections should be designed for load combinations in 4.2.2 or by the applicable building code.

ASCE/SEI 7 minimum design loads are adapted to elevated water storage tanks. The loads are for Risk Category IV defined in Table 1.5-1 of ASCE/SEI 7-10. The Category IV classification includes structures designated as essential facilities required in emergencies. The structure is considered an essential facility where the stored water is required for fire protection or in an emergency.

4.2.2 Load combinations

4.2.2.1 Factored load combinations—Load combinations are divided into two groups based on if they add to the effect of dead and water loads (Group 1) or whether they counteract the dead and water loads (Group 2). Load combinations are further divided into those required by **ACI 350** for structure components in contact with stored water and those required by **ACI 318** for structure components not in contact with the stored water.

Load factors and load combinations for the strength design method should follow recommendations (a) and (b):

(a) Factored load combinations for structure components in contact with stored water should conform to ACI 350.

(b) Factored load combinations for structural components not in contact with stored water should be

i. Group 1 load combinations where structural effects of applied loads are cumulative

$$U1.1 = 1.4(D + F)$$

$$U1.2 = 1.2(D + F + G) + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) + 1.2T$$

$$U1.3 = 1.2(D + F + G) + 1.0W + 0.5L + 0.5S + 1.2T$$

$$U1.4 = 1.2(D + F + G) + 1.6(L_r \text{ or } S \text{ or } R) + 0.5L + 1.2T$$

$$U1.5 = 1.2(D + F + G) + 1.0E + 0.5L + 0.2S + 1.2T$$

ii. Group 2 load combinations where D or F reduce the effect of W or E

$$U2.1 = 0.9D + 1.0F + 1.0E$$

$$U2.2 = 0.9D + 1.0E$$

$$U2.3 = 0.9D + 1.0W$$

Load Combination U2.1 does not conform to ACI 318 because the entire weight of the acting fluid participates in the development of the seismic load. Unlike the dead load, which is not precisely known, the fluid weight is well defined.

4.2.2.2 Service load combinations—Service load combinations should follow recommendations (a) and (b):

(a) Load combinations for structure components in contact with stored water should conform to ACI 350.

(b) Service load combinations for structure components not in contact with stored water.

Group 1 load combinations where structural effects of applied loads are cumulative

i. Load combination

$$S1.1 = D + F$$

$$S1.2 = D + F + G + (L_r \text{ or } S \text{ or } R) + L + T$$

$$S1.3 = D + F + G + 0.6W + L + (L_r \text{ or } S \text{ or } R)$$

$$S1.4 = D + F + G + 0.7E + L + (L_r \text{ or } S \text{ or } R)$$

(c) Group 2 load combinations where D or F reduce effect of W or E

i. Load combination

$$S2.1 = 0.6D + 0.6W$$

$$S2.2 = 0.6D + 0.7E$$

$$S2.3 = 0.9D + 0.9F + 0.7E$$

Load Combination S1.1 is the basic long-term load combination used to check serviceability requirements such as deflections, concrete cracking, and foundation settlement. The structural effects T of creep, shrinkage, or temperature effects are usually not significant. The effect of differential settlement that causes tilt of the entire structure is included as load G defined in 4.2.9. Load Combination S2.3 includes a factor of 0.9 on dead and fluid loads due to their well-defined nature on water storage tank structures.

4.2.3 Dead loads—The dead loads include the weight of structure elements, access ladders and platforms, and all structure-supported appurtenances. The unit weight of materials used to determine dead loads should be as follows:

(a) Reinforced concrete: 150 lb/ft³ (2400 kg/m³)

(b) Steel: 490 lb/ft³ (7850 kg/m³)

(c) Aluminum: 165 lb/ft³ (2650 kg/m³)

4.2.4 Water load—The water load is the weight of the water in the tank vessel, at levels varying from empty to the overflow level, calculated at 62.4 lb/ft³ (1000 kg/m³), and the lateral pressure on the walls of the container.

4.2.4.1 Equivalent weights of accelerated water—For use in seismic analyses in 4.2.8.2(a) and 4.2.8.2(b), the equivalent weights of the convective (sloshing) component and the impulsive component of the water in circular cylindrical elevated tank vessels should be computed in accordance with **ACI 350.3**.

For geometries of tank vessels other than circular cylindrical, fluid-structure interaction analysis of the stored water may be conducted to determine the equivalent weights of the impulsive and convective components of the stored water, or the fluid-structure interaction analysis may be integrated with the modal analysis of 4.2.8.2(b).

A close approximation for impulsive and convective masses of axisymmetric, other than cylindrical, tanks may be obtained from an equivalent cylindrical tank that has the same free surface diameter as the axisymmetric tank and a depth that results in the same volume as the axisymmetric tank.

4.2.5 Live loads—Live loads include a uniformly distributed load specified in the local building code with a commonly-used minimum value of 12 lb/ft² (0.58 kPa) over the horizontal projected area of the tank vessel roof extending to the eave line, and the live loads on the access stairs and platforms of 60 lb/ft² (2.87 kPa). Where equipment is supported on platforms, an additional live load of 40 lb/ft² (1.92 kPa) should be included. A concentrated load of 1000 lb (4.45 kN), not in combination with the distributed live load, should be included on stairs and platforms.

4.2.6 Wind load

4.2.6.1 Scope—This section covers determination of minimum factored wind forces, based on Chapter 29 of **ASCE/SEI 7-10** for Risk Category III and IV structures with Exposure Categories C and D, for use in load combinations in 4.2.2. Larger factored wind loads should be used where required by the applicable building code. Refer to Table A1 for factored design wind pressures in tabular form.

The wind forces considered herein are for rigid structures with fundamental periods less than 1 second, as defined in ASCE/SEI 7-10. The potential for across-wind excitation or flutter should be investigated for tall slender tanks with fundamental periods of 1 second or greater.

4.2.6.2 Wind force—The factored wind force W acting on the structure is the sum of wind forces F_z acting on any portion of the structure defined by

$$F_z = p_z A_f \quad (4.2.6.2)$$

4.2.6.3 Lateral wind pressure—The value of lateral pressure p_z acting at height z is

$$p_z = 0.00256 G_w K_z C_f V_b^2 \text{ (lb/ft}^2\text{)} \quad (4.2.6.3a)$$

where G_w is based on ASCE/SEI 7-10 Section 26.9 and equals 0.85 for structure period T_f of 1.0 second or less and 1.0 for structure period T_f greater than 1.0 second; K_z is based on ASCE/SEI 7-10 Section 29.3, Table 29.3-1, for exposure categories defined in ASCE/SEI 7-10 Section 26.7; and V_b is based on ASCE/SEI 7-10 Section 6.5.4, Fig. 6-1.

Force coefficient C_f is

$$\begin{aligned} C_f &= 0.5 + (h/d_w - 1)60 \\ 0.5 &\leq C_f \leq 0.6 \end{aligned} \quad (4.2.6.3b)$$

The height h should not be less than height from grade to the top of vessel shell. Diameter d_w should be taken as the maximum vessel diameter or, more precisely, as weighted average of the vessel and shaft diameters with height.

The equation for wind force coefficient herein is derived from ASCE/SEI 7-10 Section 26.5.1, Fig. 26.5-1B, for moderately smooth, round cross sections with $d_w/q_z > 2.5$, and having height to maximum diameter ratios in the range of 1 to 7.

The exposure coefficient K_z is

$$K_z = 2.01(z/z_g)^{2/\alpha} \text{ for } z_{15} \leq z \leq z_g \quad (4.2.6.3c)$$

Table 4.2.6.3—Values of α and z_g

Exposure category	α	z_g , ft (m)
B	7.0	1200 (365.76)
C	9.5	900 (274.32)
D	11.5	700 (213.36)

$$K_z = 2.01(z_{15}/z_g)^{2/\alpha} \text{ for } z < z_{15} \quad (4.2.6.3d)$$

where $z_{15} = 15$ ft (4.6 m), and values of α and z_g are as listed in Table 4.2.6.3.

4.2.6.4 Wind pressure on dome roof—ASCE/SEI 7-10 Fig. 27.42 may be used to define wind pressure distribution on spherical dome roofs of elevated tanks. Other pressure distributions may be used if adequately verified, such as by wind tunnel test studies.

4.2.7 Snow load

4.2.7.1 General scope—This section covers determination of snow load, based on ASCE/SEI 7-10 using Category IV for essential facilities, Table 1.5-1; $I = 1.2$ for Category IV structures, Table 1.5-2; and $C_t = 1.2$ for unheated structures, Table 7-3. This snow load is for use in load combinations in 4.2.2. Larger snow loads should be used where required by the applicable building code. The recommendations of 4.2.7 are applicable to cone, conical, and curved roofs concave downward without steps or abrupt changes in elevation.

4.2.7.2 Roof snow load—The unfactored balanced snow load acting on the structure is the uniformly distributed snow load p_s acting on any portion of a roof times the horizontal projected area on which p_s acts. The uniformly distributed snow load p_s is the larger value determined in the following.

4.2.7.2.1 The balanced snow load on roofs with a roof slope less than or equal to 37.5 degrees should be calculated from

$$p_s = C_{es} p_g \quad (4.2.7.2.1a)$$

where p_g is described and calculated from ASCE/SEI 7-10 Fig. 7-1, or Table 3.1.2 of this guide; and C_{es} is described and calculated from ASCE/SEI 7-10 Table 7-2, and Section 6.5.6.3.

The balanced snow load on roofs with a roof slope θ_r greater than 37.5 degrees, but not exceeding 70 degrees, should be calculated from

$$p_s = C_{es} p_g (70 - \theta_r) / 32.5 \quad (4.2.7.2.1b)$$

Roofs with a roof slope $\theta_r > 70$ degrees should be considered snow free.

4.2.7.2.2 The minimum snow load acting on cone roofs with roof slope θ_r less than 15 degrees and curved and conical roofs with roof slope θ_r less than 10 degrees should be calculated from Eq. (4.2.7.2.2a) or (4.2.7.2.2b)

$$p_s = 1.2 p_g \text{ for } p_g \leq 20 \text{ lb/ft}^2 \text{ (0.96 kPa)} \quad (4.2.7.2.2a)$$

$$p_s = 24 \text{ (1.15 kPa)} \text{ for } p_g > 20 \text{ lb/ft}^2 \text{ (0.96 kPa)} \quad (4.2.7.2.2b)$$

4.2.7.3 Rain-on-snow surcharge—A location with less than 20 lb/ft² (0.96 kPa), the rain-on-snow surcharge p_r is 5 lb/ft²

(0.24 kPa) for roof slopes less than $W_r/50$ (in degrees), where W_r is the horizontal distance in ft (m) from eaves to center of cone, conical, or curved roofs.

4.2.8 Earthquake load—Earthquake load should be in accordance with **ASCE/SEI 7-10** Chapters 11, 14, 15, 19, 20, and 22. The minimum seismic forces recommended in this guide are factored loads intended to be used with the strength design load combinations of 4.2.2.1.

Elevated tanks covered by this guide behave basically as single-degree-of-freedom systems that respond primarily to the fundamental frequency of vibration and may not need sophisticated analysis techniques for determining seismic forces.

Alternative procedures that may be used for analysis include:

- (a) Modal analysis using solution to the equations of motion
- (b) Modal analysis using the response spectrum technique
- (c) Finite element time-history analysis using modal superposition or the direct integration method

4.2.8.1 The procedure for determining design earthquake ground acceleration and response spectra is as follows:

(1) The site should be classified in accordance with site class definitions of **ASCE/SEI 7-10** Section 11.4.2.

(2) The maximum considered earthquake spectral response acceleration at short periods, S_s , and at 1 second, S_1 , should be determined from **ASCE/SEI 7-10** Fig. 22-1 through Fig. 22-14.

(3) The acceleration-based site coefficient F_a and the velocity-based site coefficient F_v should be determined from **ASCE/SEI 7-10** Tables 11.4-1 and Table 11.4-2, respectively.

(4) The design spectral response accelerations for short periods, S_{DS} , and for 1-second period, S_{D1} , should be calculated from

$$S_{DS} = (2/3)F_a S_s \quad (4.2.8.1a)$$

$$S_{D1} = (2/3)F_v S_1 \quad (4.2.8.1b)$$

(5) The design response spectrum for lateral acceleration should be constructed in accordance with **ASCE/SEI 7-10** Section 11.4.5.

(6) For the vertical acceleration design spectrum, two-thirds of the design spectrum for lateral loads should be used. Refer to 4.2.8.2 for combining effects due to lateral and vertical accelerations.

The procedure for determining design earthquake ground acceleration and response spectra follows **ASCE/SEI 7-10** except for the last item in the procedure. The recommendation of Item (6) of the procedure to include consideration of vertical acceleration in the design follows the recommendation of **FEMA P-750**. The discussion in **FEMA P-750**, Chapter 23 Commentary, of the vertical response spectrum refers to the two-thirds of the horizontal acceleration spectrum.

4.2.8.2 Structural analysis procedures—The seismic base shear, seismic overturning moment at the base, and the seismic shear and overturning moment induced at any level in the elevated tank above its base may be computed using the equivalent lateral force procedure in accordance with (a) as follows of this section or the modal response spectrum analysis procedure in accordance with (b) as follows of this

section. **ASCE/SEI 7-10** Chapter 19 permits the reduction due to soil-structure interaction.

The modal analysis procedure may also be used to compute the response of the elevated tank to vertical seismic accelerations. The combined vertical and lateral seismic response effects in Load Combinations U1.5, U2.1, and U2.2 in 4.2.2.1 and Load Combinations S1.4, S2.2, and S2.3 in 4.2.2.2 should be in accordance with **Newmark and Hall (1982)** as follows:

(a) $E = 1.0 \times (\text{response to lateral acceleration}) + 1.0 \times (\text{response to vertical acceleration})$

(b) $E = 0.3 \times (\text{response to lateral acceleration}) + 1.0 \times (\text{response to vertical acceleration})$

The response to vertical acceleration in the first combination (a) should be calculated with $0.2S_{DS}$ in accordance with **ASCE/SEI 7-10**. The response to vertical acceleration in combination (b) should be calculated with two-thirds horizontal acceleration as discussed in **FEMA P-750** Chapter 23 Commentary.

The first combination of lateral and vertical seismic acceleration effects can be expected to result in maximum base shear and overturning moment, and maximum shear in the midsurface of the wall of all elevated tank components. The second combination can be expected to result in maximum hoop stress in the tank vessel, compression in domed floors, and bending in flat slab floors of composite steel-concrete tanks. The meridional compression stress in the tank vessel, the meridional compression stress in the transition of concrete tanks, and the vertical stress in the pedestal should be evaluated with both combinations.

4.2.8.2.1 The equivalent lateral force procedure is as follows:

(a) Base is the level at which horizontal ground motions are considered to be imparted to the structure. Base level is bottom of foundation, except it may be considered to be midheight of the pile cap where piles are rigidly connected to the pile cap.

(b) The seismic base shear V in a given direction should be determined from

$$V = C_s W_e \quad (4.2.8.2.1a)$$

where C_s is in accordance with Item (c) as follows of this section, and W_e is in accordance with Item (d) as follows of this section.

(c) The seismic response coefficient C_s should be computed from

$$C_s = S_{DS} I / R \quad (4.2.8.2.1b)$$

Alternatively, the seismic response coefficient C_s need not be greater than

$$C_s = S_{D1} I / T_p R \quad (4.2.8.2.1c)$$

but should not be taken less than

$$C_s = 0.044 S_{T1} I$$

where T_f is determined in (e) as follows of this section, and R and I are in accordance with ASCE/SEI 7-10 Tables 15.4-2 and 1.5-2, respectively, or as required by the applicable building code.

(d) The effective weight W_e should include the total dead weight of the elevated tank in accordance with 4.2.3, the effective weight of the water in accordance with 4.2.4.1, 25 percent of live load, and 20 percent of roof snow load where flat roof snow load exceeds 30 lb/ft² (1.4 kPa).

(e) The fundamental period T_f should be established using the effective weight in (d) of this section and stiffness characteristics of the resisting elements in a properly substantiated analysis.

Rayleigh's quotient may be used

$$T_f = 2\pi\sqrt{\sum(W_i\delta_i^2)/g\sum(F_i\delta_i)} \quad (4.2.8.2.1e)$$

Alternatively, for the composite steel-concrete tank, T_f may be approximated from

$$T_f = (0.16\ell_g/d_w)\sqrt{\ell_g/g} \quad (4.2.8.2.1f)$$

The derivation of Eq. (4.2.8.2.1f) may be found in Appendix A of this guide.

For the all-concrete tank, T_f may be approximated from

$$T_f = 2\pi\sqrt{\alpha_f W \ell / 12 E_c g} \quad (4.2.8.2.1g)$$

where $\alpha_f = (L_p^3/3 + L_p^2 L_t + L_p L_t^2)/I_c + 4.68 L_p/A_c + L_t^3/3 I_t + 4.68 L_t/I_t$.

The expression for α_f in Eq. (4.2.8.2.1g) is proportional to the tip deflection of a tip-loaded cantilever beam with section properties that are constant over a length near the tip and different from those over a length near the root. Both bending and shear deflections are included. The 4.68 factor for the shear terms is based on a Poisson's ratio of 0.17 for concrete and a shear shape factor of 2.0 for a thin cylindrical section.

(e) Vertical distribution of the lateral seismic force F_x induced at any level should be determined from

$$F_x = C_{vx} V \quad (4.2.8.2.1h)$$

and

$$C_{vx} = \frac{W_x h_x^k}{\sum_{i=1}^n W_i h_i^k} \quad (4.2.8.2.1i)$$

where k is the exponent related to the structure period as follows: for structures having a period of 0.5 seconds or less, $k = 1$; for structures having a period of 2.5 seconds or more, $k = 2$; and for structures having a period between 0.5 and 2.5 seconds, k should be 2 or determined by linear interpolation between 1 and 2.

When the dead load is less than 25 percent of the total load, the total lateral seismic force can be distributed over

the height of the structure in proportion to the structure weight. The vertical distribution factor is then

$$C_{vx} = \frac{W_x}{\sum_{i=1}^n W_i} \quad (4.2.8.2.1j)$$

Equation (4.2.8.2.1j) is a simplification that considers the seismic force distribution to be proportional to the vertical distribution of the structure's weight. The results differ only slightly when most of the structure mass is contained in the stored water, which is the case for most elevated tanks. Where the dead load exceeds approximately 25 percent of the total weight, Eq. (4.2.8.2.1i) should be used. The simplest and most conservative approach is to consider the entire structure mass located at a single level, the centroid of the stored water. An analysis that considers the individual mass of stored water, vessel structure, and pedestal is usually sufficient for evaluating lateral seismic forces.

(f) The lateral seismic shear V_x acting at any level of the structure is determined by

$$V_x = \sum_{i=1}^x F_i \quad (4.2.8.2.1k)$$

where F_i is from the top of the structure to the level under consideration.

(g) The structure should be designed to resist overturning effects caused by the seismic forces determined in (e) of this section. At any level, the increment of overturning moment in the level under consideration should be distributed to the various vertical force-resisting elements in the same proportion as the distribution of the horizontal shears to those elements.

The overturning moment at level x , M_x , should be determined from

$$M_x = \sum_{i=1}^x F_i (h_i - h_x) \quad (4.2.8.2.1l)$$

(h) The design should include an accidental torsional moment caused by an assumed displacement of the mass from its actual location by a distance equal to 5 percent of the diameter of the elevated tank component. Torsional effects may be ignored when the torsional shear is less than 5 percent of the shear strength determined in 4.3.10.6.

The torsional moment requirement is equivalent to increasing the shear stress in the pedestal wall by 5 percent at sections where there are no openings. Design of the pedestal wall is rarely controlled by horizontal shear stress and, to simplify calculations, the torsional moment may be neglected when it results in in-plane shear less than 5 percent of the shear strength of the pedestal wall per unit length of circumference. For Seismic Design Categories D through F structures with large openings, torsional effects may be significant and should be included.

4.2.8.2.2 The modal response spectrum analysis is as follows:

(a) A mathematical model of the elevated tank, including the tank vessel, pedestal, and foundation, should be created