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Simplified design of prestressed concrete tanks for potable water

Conception simplifiée du réservoir pour l'eau potable en béton pré-armé

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Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

The procedures used to develop this document and those intended for its further maintenance are described in the ISO/IEC Directives, Part 1. In particular, the different approval criteria needed for the different types of ISO documents should be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see www.iso.org/directives).

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights. Details of any patent rights identified during the development of the document will be in the Introduction and/or on the ISO list of patent declarations received (see www.iso.org/patents).

Any trade name used in this document is information given for the convenience of users and does not constitute an endorsement.

For an explanation on the voluntary nature of standards, the meaning of ISO specific terms and expressions related to conformity assessment, as well as information about ISO's adherence to the World Trade Organization (WTO) principles in the Technical Barriers to Trade (TBT), see the following URL: www.iso.org/iso/foreword.html.

This document was prepared by Technical Committee ISO/TC 71, *Concrete, reinforced concrete and prestressed concrete*, Subcommittee SC 5, *Simplified design standard for concrete structures*.

Introduction

The aim of this document is to provide rules for the design and construction of prestressed concrete water tanks to be built in less-developed areas of the world. The design rules are based on simplified worldwide-accepted strength models. This document is self-contained; therefore actions (loads) and simplified analysis procedures are included, as well as minimum acceptable construction practice guidelines.

A great effort was made to include self-explanatory tables, graphics and design aids to simplify the use of this document and provide procedures. Notwithstanding, the economic implications of the conservatism inherent in approximate procedures as a substitution to sound and experienced engineering should be a matter of concern to the designer who employs this document and to the owner who hires him.

A prestressed concrete tank for potable water generally comprises the roof, wall and base slab. The roof is made to entirely cover the top of the cylindrical wall so as to protect the water from contamination with rainwater, etc. In many cases, it is made in the form of a dome shaped like a convex disc cut off from a sphere. The wall is a vertical cylinder that forms a container for water in combination with the flat disc base slab. Normally, only the wall of a prestressed concrete water tank is made with prestressed concrete, while the roof and base slab are made with reinforced concrete. Prestress is generally applied to the wall using prestressing steel in the vertical and circumferential directions, but in some cases prestress is applied only to the circumferential direction. For this reason, this document defines a prestressed concrete cylindrical tank as a structure having prestressing steel at least in the circumferential direction of the wall to apply prestress, so as to cover both types. Therefore, the roof, base slab and wall in the vertical direction may not necessarily be of prestressed concrete construction but may be of reinforced concrete construction.

A prestressed concrete water tank construction is generally adopted to preserve a water storage facility with the aim of preventing severe secondary disasters and allowing the standing water to be used as an emergency water supply. For this reason, it is required to be designed as a rule as a high degree of importance.

The minimum dimensional provisions contained in this document are intended to account for undesirable side effects that will require more sophisticated analysis and design procedures. Material and construction provisions are aimed at site-mixed concrete, as well as ready-mixed concrete and steel of the minimum available strength grades.

The earthquake-resistance provisions are included to account for the fact that numerous underdeveloped regions of the world occur in earthquake-prone areas. The earthquake resistance is based upon the employment of structural concrete walls (shear walls) that limit the lateral deformations of the structure and provide for its lateral strength.

This document contains provisions that can be modified by the National Standards Body due to local design and construction requirements and practices. The specifications that can be modified are indicated using [“boxed values”]. The National Standards Body is expected to review the “boxed values” and may substitute alternative definitive values for these elements for use in the national application of this document.

Simplified design of prestressed concrete tanks for potable water

1 Scope

This document provides guidelines for the planning, design and construction of a cylindrical tank constructed on the ground with prestressed concrete (PC) for use with potable water tank.

This document is applicable to PC tanks for potable water with a capacity of 30 000 m^3 or less and the diameter-to-height ratio (D/H) from 1,0 to 3,0.

NOTE When designing and constructing a tank not covered by this document (reinforced concrete tanks, underground tanks, elevated tanks, etc.), a designer can refer to this document for common elements where possible.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 1920-3, *Testing of concrete — Part 3: Making and curing test specimens*

ISO 1920-4, *Testing of concrete — Part 4: Strength of hardened concrete*

ISO 6934-1, *Steel for the prestressing of concrete — Part 1: General requirements*

ISO 6934-2, *Steel for the prestressing of concrete — Part 2: Cold-drawn wire*

ISO 6934-3, *Steel for the prestressing of concrete — Part 3: Quenched and tempered wire*

ISO 6934-4, *Steel for the prestressing of concrete — Part 4: Strand*

ISO 6934-5, *Steel for the prestressing of concrete — Part 5: Hot-rolled steel bars with or without subsequent processing*

ISO 6935-1, *Steel for the reinforcement of concrete — Part 1: Plain bars*

ISO 6935-2, *Steel for the reinforcement of concrete — Part 2: Ribbed bars*

ISO 6935-3, *Steel for the reinforcement of concrete — Part 3: Welded fabric*

ISO 12439, *Mixing water for concrete*

ISO 14654, *Epoxy-coated steel for the reinforcement of concrete*

ISO 14824-3, *Grout for prestressing tendons — Part 3: Test methods*

3 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

— ISO Online browsing platform: available at https://www.iso.org/obp

— IEC Electropedia: available at https://www.electropedia.org/

3.1
bending analysis
method for determining the membrane force and bending moment in consideration of the boundary conditions at the base of the dome

3.2
clearance
distance between the designed high-water level and the upper edge of the tank wall

3.3
convective pressure
water pressure produced by oscillation of the water

3.4
cylindrical prestressed concrete tank
concrete tank comprising the roof, cylindrical wall and base slab, for which prestressing steel is provided at least in the circumferential direction to apply prestress

3.5
disc part
part other than the ring plate of the base slab that resists bending moments

3.6
dome ring
circular beam provided along the base of the roof of a spherical or other shape of the dome to control radial displacement at the base of the roof

3.7
dynamic water pressure
water pressure due to the effect of an earthquake

3.8
embedded system
system of applying prestress, whereby circumferential prestressing steel is provided within concrete members

3.9
fixed support
wall-bottom connection whereby the rotation or horizontal displacement of the wall with respect to the bottom is not allowed

3.10
foundation slab
reinforced concrete or prestressed concrete slab provided in contact with the bottom surface of the base slab

3.11
freely sliding support
wall-bottom connection, whereby the rotation and horizontal displacement of the wall with respect to the bottom are allowed

3.12
hinged support
wall-bottom connection, whereby the rotation of the wall with respect to the bottom is allowed

3.13
hoop tension
circumferential axial tensile force generated by such loads as water pressure

3.14
horizontal thrust
horizontal component of the axial force in the meridian direction of the dome at the base of the dome

3.15
Housner method
conventional approximate analysis method for liquid vibration proposed by G. W. Housner

3.16
imposed load
load of portions not included in the design calculation as structural members and load applied to the roof for such purposes as inspection

3.17
impulsive pressure
dynamic water pressure (3.7) in response to short-period components of an earthquake and water pressure associated with inertial force produced by accelerations of the tank wall and directly proportional to these accelerations

3.18
inertia force
force given by the product of the weight of a body and the design seismic intensity

3.19
in-plane shear force
shear force that acts parallel to the shell surface

3.20
membrane floor
part other than the ring plate of the base slab that does not resist bending moments

3.21
membrane force
in-plane axial force of a shell structure

3.22
out-of-plane shear force
shear force that acts at a right angle to the shell surface

3.23
particular load
special load that acts depending on the natural conditions of the tank construction site

Note 1 to entry: Particular load is judged as a *primary load* (3.25) or a *subsidiary load* (3.31) on a case-by-case basis.

3.24
pilaster
rectangular projections from the tank wall along its generatrix lines for anchoring circumferential prestressing steel

3.25
primary load
load that constantly acts

3.26
ring plate
peripheral part of the base slab for transmitting forces primarily from the tank wall to the ground

3.27
sloshing
vibration of a solid oscillating surface generated in response to relatively long-period components of an earthquake

3.28
solid mass of water
equivalent weight of water to produce the impulsive force on the tank wall

Note 1 to entry: It is assumed to be fastened rigidly to the tank wall.

3.29
solid oscillating mass
equivalent oscillating weight to produce the convective force on the wall

Note 1 to entry: It is assumed to be fastened to the tank wall by spring.

3.30
spherical dome
curved shell in the form of a part of a sphere cut off by a plane

3.31
subsidiary load
load that rarely acts

3.32
tank empty condition
state in which no water is present in the tank

3.33
tank full condition
state in which the water level in the tank reaches the design high water level

3.34
velocity potential method
method for a theoretical solution to irrotational vibration of a non-compressive and non-viscous fluid

3.35
waterstop
plate inserted in joints between concrete lifts and the wall-bottom joints for waterstopping

4 Symbols

A projection area

A_b area of concrete subjected to bearing load

A_c total area of concrete surface

A_d surface area of the dome

A_{EP} area subjected to the effect of anchorage set

A_i cross-sectional area of element i (member between nodes i and i + 1)

A_p cross-sectional area of prestressing steel

A_s cross-sectional area of tensile reinforcement

b member width

C wind force coefficient

C_e earth pressure coefficient

C_s structure characteristic coefficient

C_z correction factor by region

D diameter of the tank

D_{he} correction factor dependent on damping constant

D_i flexural stiffness of node i $\left(= \dfrac{E t_i^3}{12\left(1 - \nu^2\right)} \right)$

D_p pile diameter

D_η response reduction ratio due to plastic deformability

E elastic modulus

E_c elastic modulus of concrete

E_p elastic modulus of prestressing steel

E_s elastic modulus of steel reinforcement

f'_{cd} design compressive strength of concrete

f'_{ck} characteristic compressive strength of concrete

f_{pu} tensile strength of prestressing steel

f_{py} yield strength of prestressing steel

f_{sy} yield strength of steel reinforcement

f_{ud} design tensile strength of prestressing steel

f_{yd} design yield strength of steel reinforcement and structural steel

g gravitational acceleration

g_0 uniform pressure

H height or total water depth of the tank

H_G distance from the bottom of the tank wall to the point of action of the dome inertia force or other partial weight inertia force

H_h length of thickened wall (haunch height)

H_i thickness of i-th stratum

H_s total height of earth pressure action

H_t horizontal thrust

H_x water depth at an arbitrary point

h height from the ground surface

h_{rE} distance from tank bottom to a solid mass point when dynamic water pressure on base slab is neglected

h_{rI} distance from tank bottom to a solid mass point when dynamic water pressure on base slab is considered

h_{sE} distance from tank bottom to solid oscillating mass point when dynamic water pressure on base lab is neglected

h_{sI} distance from tank bottom to solid oscillating mass point when dynamic water pressure on base slab is considered

h_{th} virtual thickness of member

I_i second moment of area of element i (member between nodes i and $i + 1$)

J_1 vessel function

K flexural stiffness $\left(=\dfrac{E t^3}{12\left(1-\nu^2\right)}\right)$

K_h design horizontal seismic coefficient

K_{ho1} standard horizontal seismic coefficient of structure for Level 1 ground motion

K_{ho2} standard horizontal seismic coefficient of structure for Level 2 ground motion

K_{h1} design horizontal seismic coefficient for Level 1 ground motion

K_{h2} design horizontal seismic coefficient for Level 2 ground motion

K_v vertical subgrade reaction modulus

K_{vi} vertical spring constants of node i

$K_{\theta i}$ rotational spring constants of node i

k spring constant of bearing

k_α coefficient incorporating the characteristics of foundations

k_β coefficient incorporating the characteristics of base slab

L_{rp} ring plate width

L_w water depth

l length from the tension end of prestressing steel to the design cross-section

l_d basic development length

l_{max} maximum spacing of prestressing steel

l_p length of prestressing steel

l_1 distance from the top of wall to the beginning point of action of distributed load

l_2 distance from the top of wall to the end point of action of distributed load

Δl set length

M_a corrected vertical bending moment

M_d vertical bending moment of member

M_e vertical bending moment at the bottom of the tank wall generated by changes in the curvature radius of the tank wall

M_{ud} design flexural fracture capacity

M_x vertical bending moment

M_{xi} bending moment at node i determined by planar frame analysis

$M_{x\phi}$ torsional moment

M_0 restraining moment at the bottom of the tank wall

M_{0c} vertical bending moment at the bottom of the tank wall with a constant thickness

M_{0e} vertical bending moment at the bottom of the tank wall generated by curvature changes when the wall thickness is constant at t

M_{0f} vertical bending moment at bottom of the tank wall

M_{0h} vertical bending moment at the bottom of the tank wall incorporating increases in the wall bottom thickness

$M_{0T}(x)$ overturning moment at a distance x from the top of the tank wall

$M_{0\nu}$ vertical bending moment at the bottom of the tank wall generated by the effect of Poisson's ratio due to vertical prestress, ν, when the wall thickness is constant

$\overline{M_x}$ vertical bending moment obtained from axisymmetric analysis under equivalent load

M_ϕ circumferential bending moment

$M_{\phi x}$ torsional moment

$M_{\theta i}$ bending moment per unit length in the circumferential directions

N_x axial force in the vertical direction

$N_{xo}(x)$ vertical axial force at distance from the bottom of the tank wall

$N_{x\phi}$ in-plane shear force

N_ϕ axial force in the circumferential direction

$N_{\phi d}$ membrane force per unit length of the dome in the meridian direction

$N_{\phi x}$ in-plane shear force

$N_{\theta d}$ membrane force per unit length of the dome in the parallel direction

n elastic modulus ratio ($=E_p/E_c$)

P prestressing force in the vertical direction

$P_{br}(r)$ maximum impulsive pressure that acts on the base slab

$P_{bs}(r)$ maximum vibration pressure that acts on the base slab

P_i tensile force of prestressing steel at the jack position

P_l dynamic water pressure at lower edge

P_r horizontal force acting on the solid mass point

P_{ri} concentrated load acting on node i (including moment load)

P_s horizontal force acting on the s-th soild oscillating mass point

P_{sh} total earth pressure

P_{si} concentrated load per unit length acting on node i

P_t tensile force of prestressing steel at jack position after considering the set length

P_u dynamic water pressure at upper edge

P_w wind load

$P_{wr}(\xi)$ maximum impulsive pressure that acts on tank wall

$P_{ws}(\xi)$ maximum convective pressure that acts on tank wall

P_x tensile force of prestressing steel at design cross-section

P_{0l} impulsive pressure at the bottom end

P_{1l} convective pressure at the bottom end

P_{1u} convective pressure at the top end

p pressure at the bottom of the tank wall

p_{sh} earth pressure

p_w hydrostatic water pressure at an arbitrary depth from the water surface

$p_o(x)$ radial component of load

Q_a load at the time of allowable displacement

Q_{cr} cracking load of member

Q_e elastic response axial tensile force

Q_{he} circumferential axial tensile force of member by elastic analysis under an earthquake load calculated from design horizontal seismic coefficient

Q_{py} load at yielding of prestressing steel

Q_x out-of-plane shear force

Q_y yield load of member

Q_0 restrained shear force at the bottom of the tank wall

Q_ϕ out-of-plane shear force

q wind velocity pressure

q_d deadweight per unit area

q_l imposed load per unit area

$q_o(x)$ circumferential component of load

q_0 unit weight of liquid

R radius of the tank wall

r radius of the dome

r_h ratio of secondary stiffness after yielding to yield stiffness

r_i radius of node i (x coordinate)

S_v velocity response spectrum

T temperature difference

T_c resultant force of tensile stress generated in concrete

T_G natural period of the ground

t wall thickness

t_b base slab thickness

t_e effective age of concrete at the time of loading and calculating creep factor

t_h thickness of the bottom of the tank wall with a haunch

t_i thickness of base slab at node i

t_{min} minimum thickness of the tank wall

t_{rp} thickness of ring plate

t_0 effective age of concrete at the time of loading and calculating creep factor

u perimeter of member in contact with outdoor air

V tank volume capacity

V_{si} mean shear elastic wave velocity at i-th stratum

W total weight of contained water

W_r weight of solid mass point

W_s weight of s-th solid oscillating mass point

w_s unit weight of soil

w_x radial displacement of the tank wall

w_{xo} radial displacement obtained from axisymmetric analysis under equivalent load

w_1 strength of uniform load

w_2 strength of triangularly distributed load

x distance from the bottom of the tank wall

x_t distance from the tension edge of member to neutral axis

y_s height of the point of action of earth pressure

Z section modulus for the distance from the neutral axis to the compression edge when the tank is assumed to be a thin-wall ring

α angular change of prestressing steel

α_d half-open angle of the dome

α_e linear expansion coefficient

α_p correction factor for vertical bending moment at the bottom of the tank wall due to change in the curvature radius of the tank wall

α_r response acceleration of solid mass point

α_s response acceleration of s-th solid oscillating mass point

α_1 pressure gradient

β characteristic value of the tank wall $\left(=\sqrt[4]{\dfrac{Et}{4R^2K}}\right)$

$\beta_d(t_e - t_0)$ function for effective number of days after loading, $(t_e - t_0)$

$\beta_f(t_e)$ function for effective age of concrete, t_e, and virtual thickness of member, h_{th}

β_1 pressure at the bottom of the tank wall

γ relaxation ratio of prestressing steel

γ_b member factor

γ_i structural factor

γ_m material factor

γ_0 inner diameter-water depth ratio $(=D/H)$

ε_{cr} crack strain of member

ε_{py} yield strain of prestressing steel

ε_r circumferential response strain

ε_{ra} circumferential strain limit determined from securing of watertightness

ε_{sy} yield strain of steel reinforcement

ε_y virtual yield strain of member

ε'_c creep strain of concrete

ε'_s drying shrinkage of concrete

η effectiveness coefficient (= σ_{pe}/σ_{pt})

η_p average magnification of cumulative plastic deformation

η_r response average magnification of cumulative plastic deformation

θ counter-clockwise angle form the direction of acceleration

λ friction coefficient per unit length of prestressing steel

λ_e environmental factor

δ_y yield displacement

δ_0 cumulative plastic deformation

μ friction coefficient per radian of angle change

ν Poisson's ratio

ξ distance from the top of the tank wall

ρ unit weight of water

ρ_c unit weight of concrete

σ_c tensile stress of concrete

σ_{pe} effective tensile stress of prestressing steel

σ_{pt} tensile stress of prestressing steel immediately after prestressing

σ_{sa} tensile stress limit of reinforcement

σ'_{ba} bearing stress limit of concrete

σ'_c compressive stress of concrete

σ'_{ca} compressive stress limit of concrete

σ'_{cp} stress of concrete cross-section under sustained load at the centroid of prestressing steel

σ'_{cpg} concrete stress at the centroid of prestressing steel due to prestressing

σ'_{cpt} prestress immediately after prestressing at the centroid of prestressing steel

ΔP tensile force loss of prestressing steel due to set length of anchors

Δt number of days when the temperature is T

$\Delta\sigma_p$ tensile stress loss of prestressing steel due to elastic deformation of concrete

$\Delta\sigma_{p\gamma}$ tensile stress loss of prestressing steel due to relaxation of prestressing steel

$\Delta\sigma_{p\phi}$ tensile stress loss of prestressing steel due to creep and drying shrinkage of concrete

ϕ diameter of steel reinforcement

ϕ_d angle from the rotation axis at an arbitrary point of the dome

ϕ_i deflection angle at node i determined by planar frame analysis

ϕ_p diameter of prestressing steel

ϕ_s internal friction angle of soil

φ creep factor for concrete

φ_{d0} basic creep factor for delayed elastic strain

φ_{f0} basic creep factor for flow strain

ω_s s-th natural circular frequency

5 Design principles

To design the PC water tank safe under loads application, its construction site shall be carefully selected through investigation on topography, geology and past disaster records. It is necessary to conduct ground surveys at the stage of structural planning according to the state of the site and the type and size of the water PC tank.

Safety of members in an ordinary state shall be investigated by confirming that the stresses of concrete, prestressing steel and steel reinforcement under loads are equal to or less than the stress limits specified in Clause 8. The primary purpose of the design of a PC water tank is to secure its watertightness in service and the PC water tank designed in this manner is generally known to have sufficient bearing capacity against failure (ultimate limit state). The elastic design method is, therefore, adopted in this document with regard to safety and serviceability of members; that is, the serviceability and ultimate limit states, respectively, in an ordinary state in consideration of the following: the fact that the tensile stress limit of concrete to be prestressed is kept low so as not to cause cracking even under the action of subsidiary loads, while requiring adequate tensile reinforcement is arranged where tensile stress is generated to increase the deformability of the cross-section and the requirement for a considerably low stress limit for steel reinforcement under surfaces to come into contact with water.

Safety of members shall be investigated with respect to the most severe unfavourable combination of loads. Safety of members during an earthquake shall be investigated, as necessary, by confirming that the PC water tank can ensure the performance requirements during an earthquake. They shall be verified in relation to the combinations of ground motion levels used for seismic design and the importance degrees of the PC tank. Details are described in Clause 9.

The member force to be used for the design of members shall be calculated based on the elastic theory on the assumption that the entire cross-section of concrete is effective even with partial cracking, partly because the stress limit is specified so that the crack width would be smaller than in general concrete members in consideration of the characteristics of the PC tank. In this case, the member force may be calculated by assuming the entire cross-section of concrete to be effective while ignoring reinforcement.

Although the provisions contained in this document were established to produce, when properly employed, a prestressed concrete structure with an appropriate margin of safety, this document is not a substitute for sound and experienced engineering. In order for the resulting structure designed in accordance with these provisions to attain the intended margin of safety, this document shall be used as a whole and alternative procedures should be employed only when explicitly permitted by the provisions. In most cases, the minimum dimensional provisions in this document are considered to be equivalent to those in national design codes. Yet, the simplicity of the procedures presented in this document can have a positive economic impact on the construction of prestressed concrete structures.

The professional performing the structural design in accordance with this document should have appropriate training and a minimum of knowledge of structural mechanics, statics, strength of materials, structural analysis and prestressed concrete design and construction.

The reference design flow is given in Annex A.

NOTE An example of design calculation in accordance with this document is given in Annex E.

6 Load

6.1 General

When designing a PC water tank, consideration shall be given to the deadweight, imposed load, hydrostatic water pressure, prestressing force, effects of creep and drying shrinkage of concrete, effect of an earthquake, wind load, earth pressure, uplift pressure, etc., as necessary. The loads to be used for design shall be selected after thoroughly surveying and checking the natural conditions of the construction site of the tank. The loads given in the above requirement are classified into three categories, primary loads, subsidiary loads and particular loads, as given in Table 1.

Table 1 — Load classification

Primary load	a)	Deadweight
	b)	Imposed load
	c)	Hydrostatic pressure
	d)	Prestressing force
	e)	Effects of creep and drying shrinkage of concrete
Subsidiary load	a)	Effect of an earthquake
	b)	Effect of temperature
	c)	Wind load
Particular load	a)	Snow load
	b)	Earth pressure
	c)	Uplift pressure
	d)	Other

NOTE Among the particular loads, snow loads have often been treated as primary loads in areas of heavy snowfall.

6.2 Deadweight

When the actual density is available, that value shall be used. If not, the density according to ISO 9194 for estimating the unit weight or the unit weight given in Table 2 may be used for calculating deadweight.

Table 2 — Unit weight of concrete

Type	Unit weight (kN/m^3)
Plain concrete	23,0
Reinforced concrete	24,5
Prestressed concrete	24,5
Mortar	21,0

6.3 Imposed load

An imposed load of [0,5] kN/m^2 should be considered assuming the weight of people climbing up on the roof for inspection. When waterproof mortar, handrails, etc. are included, their actual weights are required to be added.

6.4 Hydrostatic water pressure

Hydrostatic water pressure shall be assumed to be proportional to the depth of water and act at right angle to the surfaces of the structure. Hydrostatic water pressure, p_w, is calculated by Formula (1):

$$p_w = \rho H_x \tag{1}$$

where

p_w is the hydrostatic water pressure at an arbitrary depth from the water surface;

ρ is the unit weight of water;

H_x is the water depth at an arbitrary point.

6.5 Prestress

6.5.1 General

Prestressing forces shall be applied using prestressing steel for mitigating tension in members. Water pressure causes hoop tension and bending moment in the horizontal and vertical directions, respectively, of the wall of a cylindrical tank. Also, a roof of a spherical dome type causes hoop tension at the dome ring due to the roof load including the deadweight of the dome.

a) Circumferential prestress

Hoop tension of a cylindrical tank derived from water pressure shall be counterbalanced by placing, prestressing and anchoring prestressing steel in the circumferential direction. It is advisable to determine the prestress to be applied in consideration of the effects of an earthquake and so forth.

b) Vertical prestress

In regard to bending moment in the vertical direction acting on the tank wall, prestress construction is generally adopted by placing vertical prestressing steel. Reinforced concrete construction having vertical steel reinforcement is also permitted.

c) Prestress along the dome ring

Horizontal thrust due to roof loads including the deadweight of the dome acting on the dome ring can be counterbalanced by applying a prestressing force by placing, prestressing and anchoring prestressing steel.

6.5.2 Prestressing force immediately after prestressing

The prestressing force immediately after prestressing shall be calculated in consideration of the following effects in addition to the applied tensile force.

a) Elastic deformation of concrete

The tensile stress losses of prestressing steel due to elastic deformation of concrete may be calculated by Formula (2):

$$\Delta\sigma_p = \frac{1}{2} n \sigma'_{cpg} \tag{2}$$

where

$\Delta\sigma_p$ is the tensile stress loss of prestressing steel due to elastic deformation of concrete;

n is the elastic modulus ratio, (= E_p/E_c);

E_p is the elastic modulus of prestressing steel;

E_c is the elastic modulus of concrete at the age of prestressing;

σ'_{cpg} is the concrete stress at the centroid of prestressing steel due to prestressing.

b) Friction between prestressing steel and sheath

The tensile force of the prestressing steel at the design cross-section incorporating the effect of friction between the prestressing steel and the sheath can be expressed by Formula (3):

$$P_x = P_i\, e^{-(\mu\alpha+\lambda l)} \tag{3}$$

where

P_x is the tensile force of prestressing steel at the design cross-section;

P_i is the tensile force of prestressing steel at the jack position after considering the set length;

μ is the friction coefficient per radian of angular change;

α is the angular change of prestressing steel;

λ is the friction coefficient per unit length of prestressing steel;

l is the length from the tension end of prestressing steel to the design cross-section.

Table 3 gives standard values of friction coefficients.

Table 3 — Friction coefficients

Type of prestressing steel	μ	λ
Wires and wire strands	0,30	0,004
Bars	0,30	0,003

c) Set length of anchorages

Tensile force losses of prestressing steel due to anchor setting vary depending on the prestressing method. The losses are negligible by the threaded type and button-head type. When using wedge-type anchorages, which lead to large set lengths, the resulting tensile force losses and affected ranges shall be investigated.

The tensile force loss due to anchorage setting when there is no friction between prestressing steel and sheath may be calculated by Formula (4):

$$\Delta P = \frac{E_p A_p \Delta l}{l_p} \tag{4}$$

where

ΔP is the tensile force loss due to setting of prestressing steel;

l_p is the length of prestressing steel;

Δl is the set length;

A_p is the cross-sectional area of prestressing steel;

E_p is the elastic modulus of prestressing steel.

When there is friction between prestressing steel and sheath, it is advisable to determine the tensile force loss due to anchorage setting by the diagram method as described below.

NOTE The diagram method takes the case of four pilasters shown in Figure 1 as an analogy.

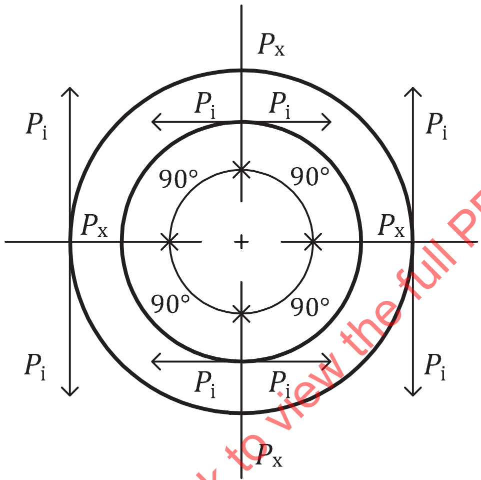


Figure 1 — Circumferential prestressing steel arrangement for four pilasters

The anchorage set length is expressed as Formula (5):

$$\Delta l = \frac{A_{EP}}{A_p E_p} \tag{5}$$

where

Δl	is the set length;
A_{EP}	is the area subjected to the effect of anchorage set;
A_p	is the cross-sectional area of prestressing steel;
E_p	is the elastic modulus of prestressing steel.

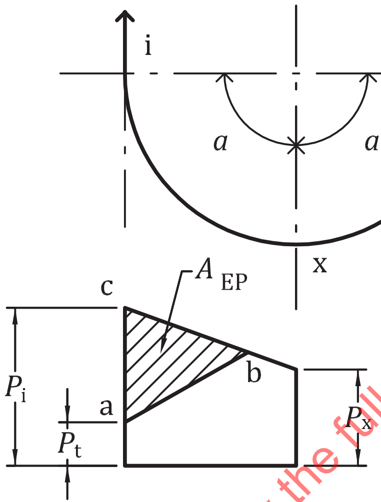


Figure 2 — Tensile force loss of prestressing steel due to setting (a)

Determine point b at which $\Delta l\, A_p\, E_p = A_{EP}$ to determine Line a-b as illustrated in Figure 2. This line expresses the distribution of the tensile forces of prestressing steel incorporating the tensile force loss due to anchorage setting. The example above shows the case where (a) the effect of anchorage setting does not reach the design cross-section. When (b) the effect reaches the boundary of the design cross-section, Figure 3 applies.

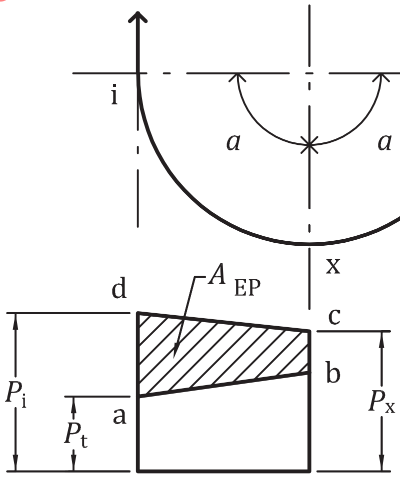


Figure 3 — Tensile force loss of prestressing steel due to setting (b)

6.5.3 Effective prestressing force

The effective prestressing force shall be calculated in consideration of the following effects in addition to the prestressing force immediately after prestressing calculated in accordance with 6.5.2.

a) Creep and drying shrinkage of concrete

The effective prestress shall be determined by calculating the tensile force loss of prestressing steel due to the creep and drying shrinkage of concrete and the relaxation of prestressing steel. The tensile stress loss of prestressing steel due to the creep and drying shrinkage of concrete may be calculated by Formula (6):

$$\Delta\sigma_{\mathrm{p}\varphi} = \frac{n\varphi\sigma'_{\mathrm{cp}} + E_{\mathrm{p}}\varepsilon'_{\mathrm{s}}}{1 + n\dfrac{\sigma'_{\mathrm{cpt}}}{\sigma_{\mathrm{pt}}}\left(1 + \dfrac{\varphi}{2}\right)} \tag{6}$$

where

$\Delta\sigma_{\mathrm{p}\varphi}$	is the tensile stress loss of prestressing steel due to creep and drying shrinkage of concrete;
n	is the elastic modulus ratio (= $E_{\mathrm{p}}/E_{\mathrm{c}}$);
E_{p}	is the elastic modulus of prestressing steel;
φ	is the creep factor of concrete;
$\varepsilon'_{\mathrm{s}}$	is the drying shrinkage of concrete;
σ'_{cp}	is the stress of concrete under sustained load at the position of prestressing steel (for a tank wall, σ'_{cp} may be regarded as prestress immediately after prestressing);
σ_{pt}	is the tensile stress of prestressing steel immediately after prestressing;
σ'_{cpt}	is the prestress immediately after prestressing at the position of prestressing steel.

b) Relaxation of prestressing steel

The tensile stress loss of prestressing steel due to relaxation of prestressing steel may be calculated by Formula (7):

$$\Delta\sigma_{\mathrm{p}^3} = \gamma\sigma_{\mathrm{pt}} \tag{7}$$

where

$\Delta\sigma_{\mathrm{p}\gamma}$	is the tensile stress loss of prestressing steel due to relaxation of prestressing steel;
γ	is the relaxation ratio of prestressing steel;
σ_{pt}	is the tensile stress of prestressing steel immediately after prestressing.

The effective tensile stress and effectiveness coefficient shall be calculated by the following Formula (8) and (9):

$$\sigma_{\mathrm{pe}} = \sigma_{\mathrm{pt}} - \Delta\sigma_{\mathrm{p}\varphi} - \Delta\sigma_{\mathrm{p}\gamma} \tag{8}$$

$$\eta = \frac{\sigma_{\mathrm{pe}}}{\sigma_{\mathrm{pt}}} \tag{9}$$

where

σ_{pe} is the effective tensile stress of prestressing steel;

σ_{pt} is the tensile stress of prestressing steel immediately after prestressing;

$\Delta\sigma_{p\varphi}$ is the tensile stress loss of prestressing steel due to creep and drying shrinkage of concrete;

$\Delta\sigma_{p\gamma}$ is the tensile stress loss in tensile stress of prestressing steel due to relaxation of prestressing steel;

η is the effectiveness coefficient.

When the effective prestress is calculated during outline design by roughly giving the tensile stress of prestressing steel immediately after prestressing, σ_{pt}, the effectiveness coefficient, η may be assumed to be 0,8 to 0,85. When judged as disadvantageous by design calculation, however, this should be precisely calculated.

6.5.4 Indeterminate forces due to prestress

When indeterminate forces are generated by prestressing forces, these shall be taken into account as necessary.

6.6 Creep and drying shrinkage of concrete

In the case where the tank is to be filled with water at an early time (within three months after completion) after completion, the effects of the creep and drying shrinkage of concrete can be neglected in the calculation of cross-sectional forces.

In the case where the completed tank is to be left unfilled for a long time, indeterminate forces may be generated at the bottom edge of the tank wall, due to, for instance, differences in the age or the progress of drying shrinkage between the base slab and tank wall. The minimum reinforcement shall be specified in consideration of this effect (see 11.4.2.3).

The effect of creep shall be incorporated when the structural system changes, such as the case where the form of supporting the bottom edge of the wall changes.

6.7 Effect of temperature

The design of a PC water tank shall incorporate the effect of uniform temperature rise/drop and the effect of temperature gap between the inside and outside of the tank wall. However, because of being slow, uniform temperature rise and drop may be considered to cause no temperature difference from one member to another.

When a PC water tank is empty, temperature differences between the inside and outside of the tank wall are marginal, and therefore, their effect may be disregarded. When the tank is filled with water, however, differences between the temperatures of water and air across the tank wall cause temperature stress. Such temperature differences vary depending on the region, water source, season, weather, etc. For these reasons, a design temperature difference (temperature gradient) of [5] °C may be adopted when no detailed data are available. Note that the effect of temperature shall be considered by generally assuming that external air temperature is higher than tank water temperature in summer and vice versa in winter.

6.8 Seismic action

The effects of an earthquake to be generally considered for the seismic design of a PC water tank shall include:

— displacement and strain of the ground during an earthquake;

— inertia forces derived from the deadweight of structures;

— earth pressure during an earthquake;

— dynamic water pressure during an earthquake;

— sloshing;

— lateral flow of the ground due to liquefaction;

— ground strain in an artificially altered sloping ground.

6.9 Wind load

The wind load, when required to be considered, shall be determined by multiplying the wind velocity pressure by the wind force coefficient and the projection area.

When investigating the bottom end stress and safety against overturning and sliding of a PC water tank having a general shape, wind load is scarcely considered in particular, because seismic force is generally greater than wind load. For a tank of particularly tall construction, the effect of wind pressure shall be investigated for a "tank empty" condition.

Wind pressure shall be calculated by Formulae (10) and (11):

$$P_{\mathrm{w}} = qCA \tag{10}$$

$$q = 0{,}6\sqrt{h} \tag{11}$$

where

P_{w} is the wind load (kN);

q is the wind velocity pressure (kN/m^2);

A is the projection area (m^2);

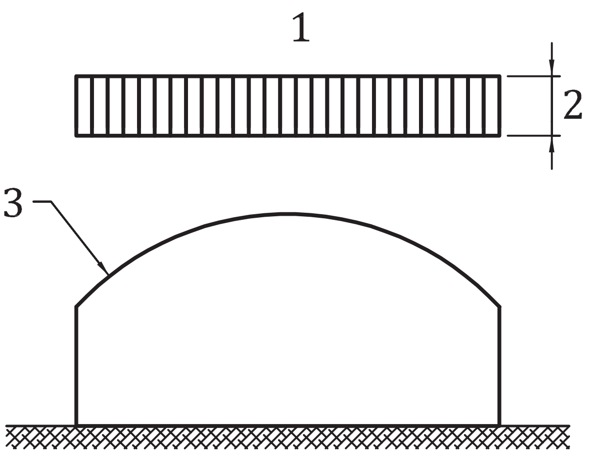
h is the height from the ground surface (m);

C is the wind force coefficient (= 0,7 for a smooth cylindrical wall;
= 1,0 for a rough cylindrical wall;
= 0,4 for a smooth spherical wall; and = 0,6 for a rough spherical wall).

6.10 Snow load

The accumulated snow load shall be determined by multiplying the unit weight of snow by the maximum snow accumulation of the region. The unit weight of snow depends on geographical latitude or altitude.

The unit weight of accumulated snow shall be established as not less than [30] N/m^2 on a normal basis for each cm of snow accumulation as shown in Figure 4. The value may be reduced to not less than [20] N/m^2 for areas with a length of snow season of less than one month.



Key

1 snow load distribution

2 maximum snow accumulation

3 dome

Figure 4 — Snow load

6.11 Earth pressure

In the case where earth pressure acts on the PC water tank, the earth pressure may be calculated by Rankine's formulae. By Rankine's formulae, the earth pressure is calculated using Formulae (12) to (15) (see Figure 5):

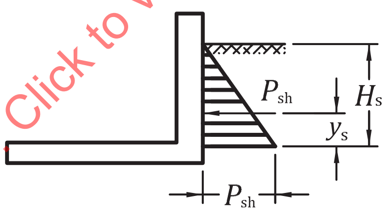


Figure 5 — Earth pressure distribution

$$p_{sh} = C_e w_s H_s \tag{12}$$

$$P_{sh} = \frac{1}{2} C_e w_s H_s^{\,2} \tag{13}$$

$$y_s = \frac{H_s}{3} \tag{14}$$

$$C_e = \frac{1 - \sin\phi_s}{1 + \sin\phi_s} \tag{15}$$

where

p_{sh} is the earth pressure;

P_{sh} is the total earth pressure force;

C_e is the earth pressure coefficient;

w_S is the unit weight of soil;

y_S is the height of the action point of earth pressure force;

H_S is the total height of earth pressure action;

ϕ_S is the internal friction angle of soil.

In case of constructing a PC water tank where the groundwater level is high, the groundwater pressure shall be considered in addition to earth pressure, which is calculated using the underwater unit weight.

Note that the compressive force due to earth pressure shall not be added to the prestressing force. When the tank is empty, however, it shall be added to the prestressing force.

6.12 Uplift pressure force

In the case where uplift pressure forces act on the bottom of the tank, investigation shall be made regarding the uplift of the whole tank and safety of the base slab.

When the base slab of the tank is located below the groundwater level or in an ill-drained ground, measures shall be taken not to cause uplift pressure on the base slab. However, when the base slab is subjected to uplift pressure, examination shall be made in regard to uplift in a “tank empty” condition and safety of the base slab.

6.13 Other loads

In the case where the tank is subjected to loads other than those described in 6.2 to 6.12, these shall be separately considered.

7 Structural analysis

7.1 Calculation of member force

The roof, wall and base slab composing a PC water tank may be analysed as thin-wall shell construction. Analysis methods, which are described in detail in Clause 11, include those using finite elements and analytical solutions.

The roof, wall and base slab constituting a PC water tank, which is generally made as an axisymmetric structure, shall be designed avoiding abrupt changes in the cross-section to the extent that it is possible, since such changes can damage the characteristics of a thin-wall structure. When an abrupt change in the cross-section is inevitable, the structural characteristics of the tank shall be thoroughly examined and the member force shall be calculated by a method suitable for the structural characteristics.

For this type of a thin-wall shell structure, examination regarding out-of-plane shear forces may generally be skipped. Therefore, when using the finite element method (FEM) for structural analysis, this structure may be modelled as thin-wall shell elements disregarding shear deformation. However, sufficient care should be exercised when modelling abrupt cross-sectional changes or junctions between structural elements by FEM.

7.2 Concrete

7.2.1 Strength

The strength of concrete to be used for design shall be based on the strength at an age of 28 days.

The characteristic strength of concrete for use in prestressed concrete members shall not be less than the following values:

— Post-tensioned: f'_{ck} = [30] MPa;

— Pre-tensioned: f'_{ck} = [35] MPa.

The characteristic strength of concrete for reinforced concrete members shall not be less than [24] MPa.

The compressive strength of concrete to which prestress can be applied to shall be not less than 1,7 times the maximum compressive stress to be generated in the concrete immediately after prestressing.

7.2.2 Modulus of elasticity

The elastic modulus used for calculating the stress of reinforced concrete members shall be 14 GPa, accordingly, the elastic modulus ratio, n, may be set at 15.

The elastic modulus used for the calculation of indeterminate forces or elastic deformation of a reinforced concrete member and the design calculation of prestressed concrete members shall be determined depending on the characteristic strength of concrete. Its examples are given in Annex D.

7.2.3 Poisson's ratio

Poisson's ratio to be used for design calculation may generally be assumed to be 0,2 in the elastic deformation range.

7.2.4 Drying shrinkage

The degree of drying shrinkage of concrete to be used for calculating the prestress loss shall be determined depending on the age of concrete at prestressing. The examples are given in Annex D.

7.2.5 Creep

The creep strain of concrete may be determined by Formula (16) on the assumption that it is proportional to the elastic strain due to the acting stress.

$$\varepsilon'_c = \varphi \frac{\sigma'_c}{E_c} \tag{16}$$

where

ε'_c is the creep strain of concrete;

φ is the creep factor of concrete;

σ'_c is the compressive stress of concrete under a sustained load;

E_c is the elastic modulus of concrete.

The creep factor for calculating the prestress loss and indeterminate force shall be determined depending on the age of concrete when a sustained load is applied. The examples are given in Annex D.

When the general creep factor is not applicable, the creep factor of concrete shall be separately determined in consideration of factors including the ambient temperature of the member, shape and size of the member cross-section and age when the load is applied to the member. The creep factor at an effective age of t_e of concrete subjected to a load at an effective age of t_0, $\varphi(t_e, t_0)$, may generally be determined by Formula (17).

$$\varphi(t_e, t_0) = \varphi_{d0}\beta_d(t_e - t_0) + \varphi_{f0}\left[\beta_f(t_e) - \beta_f(t_0)\right] \quad (17)$$

where

φ_{d0} is the basic creep factor for delayed elastic strain and is generally assumed to be [0,4];

φ_{f0} is the basic creep factor for flow strain and a value is selected from Table 4 depending on the environmental conditions;

$\beta_d(t_e - t_0)$ is a function related to the effective number of days after loading, $(t_e - t_0)$. A value read from Figure 6 may be used;

$\beta_f(t_e)$ is a function related to the effective age, t_e, and the virtual thickness of the member, h_{th}.
A value read from Figure 7 may be used;

$\beta_f(t_0)$ is function β_f related to the effective age, t_0, and the virtual thickness of the member, h_{th}.

t_0 and t_e are the effective ages (days) of concrete when concrete is loaded and when the creep factor is calculated, respectively. Values corrected according to Formula (18) may be used depending on the concrete temperature and cement type.

$$t_e \text{ or } t_0 = \alpha\,\Sigma(T + 10)\Delta t / 30 \quad (18)$$

where

α is a coefficient for the hardening rate of concrete. 2,0 and 1,0 for high early strength Portland cement and ordinary Portland cement, respectively;

Δt is the number of days when the temperature is T;

T is the temperature difference (°C).

The virtual thickness of the member, h_{th} is defined in Formula (19).

$$h_{th} = \lambda_e A_c / u \quad (19)$$

where

h_{th} is the virtual thickness of the member;

λ_e is the environmental factor (see Table 4);

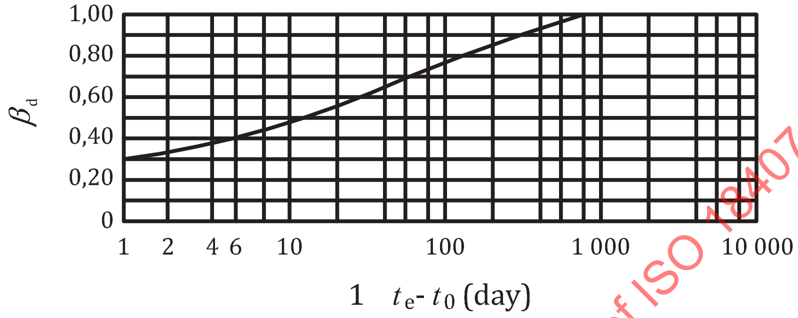
A_c is the cross-sectional area of the member;

u is the perimeter of the member in contact with ambient air.

In regard to the environmental conditions for determining φ_{f0} and h_{th}, the relative humidity in a “tank empty” condition may be assumed to be 70 %. In a “tank full” condition, the relative humidity may be assumed to be the average of underwater and 70 %. Also, the perimeter of u may be considered only on the outer side when the tank is full.

Table 4 — Basic creep factor for flow strain, φ_{f0} and λ_e

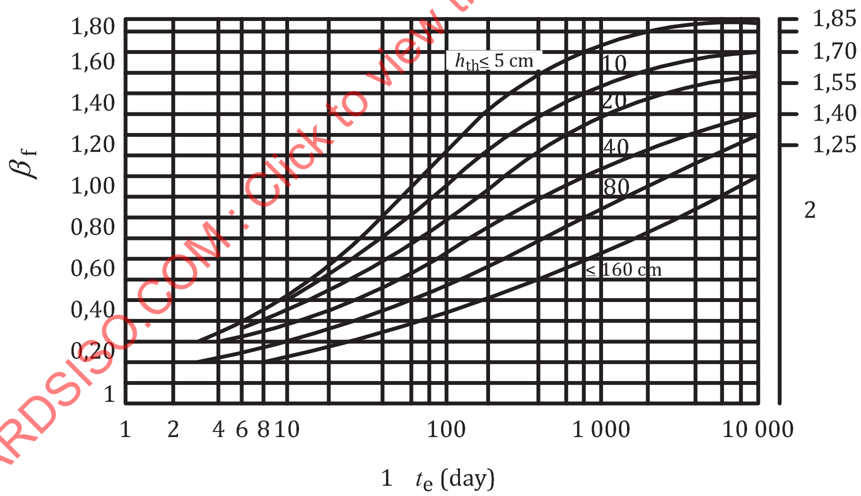
Environmental conditions	φ_{f0}	λ_e
Underwater	0,8	60
90 % R.H.	1,3	10
70 % R.H.	2,0	3
40 % R.H.	3,0	2



Key

1 effective time after loading

Figure 6 — $\beta_d(t_e - t_0)$



Key

1 effective time after loading

2 final value

Figure 7 — $\beta_f(t_e)$

7.3 Steel

7.3.1 Strength

Reinforcing bars, prestressing wire, prestressing strand, prestressing hard drawn wire, prestressing bar and welded wire fabric shall conform to the relevant ISO standards:

— Steel for reinforcement: ISO 6935-1 for plain bar and ISO 6935-2 for ribbed bar;

— Steel for prestressing steel:

  — ISO 6934-2 for cold-drawn wire;

  — ISO 6934-3 for quenched and tempered wire;

  — ISO 6934-4 for strand; and

  — ISO 6934-5 for hot rolled steel bar.

7.3.2 Modulus of elasticity

The elastic modulus of steel for design calculation may be set at 200 GPa.

7.3.3 Relaxation

As to the apparent relaxation ratio of prestressing steel for calculating the prestress loss, the values shall conform to the relevant ISO standard as presented in 7.3.1.

7.4 Calculation of tensile reinforcement

When tensile stress is generated in the cross-section of a prestressed concrete member during loading, it is necessary to place corresponding tensile reinforcement in the region under tensile stress.

For a member in which prestressing steel is not bonded with concrete, the following combination of loading action shall be considered:

(Primary load + particular load corresponding to primary load) + [1,35] × (effect of earthquake of Level 1 ground motion)

The cross-sectional area of tensile reinforcement to be placed in a cross-section of the region where tensile stress is to be generated may be calculated by Formula (20).

$$A_s = \frac{T_c}{\sigma_{sa}} \tag{20}$$

where

A_s is the cross-sectional area of tensile reinforcement;

T_c is the resultant force of tensile stress generated in concrete;

σ_{sa} is the tensile stress limit.

The stress distribution in a cross-section may be assumed as that shown in Figure 8 and the resultant tensile force, T_c is calculated by Formulae (21) and (22).

$$x_t = \frac{|\sigma_c| t}{|\sigma_c| + \sigma_c'} \tag{21}$$

$$T_c = \frac{1}{2} x_t b |\sigma_c| \tag{22}$$

where

x_t is the distance from the tension edge of member to neutral axis;

σ_c is the tensile stress of concrete;

σ'_c is the compressive stress of concrete;

b is the width of member;

t is the tank wall thickness.

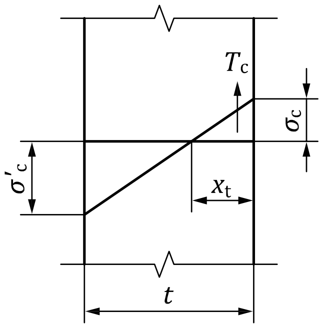


Figure 8 — Stress distribution diagram

Ribbed bars should be used for tensile reinforcement. σ_{sa} may be determined by multiplying the stress limit of reinforcement for general members by the extra coefficient given in Table 5 according to the combination of loads to be considered. Since no tensile-crack-width-controlling effect is expected from prestressing steel placed near the centroid of a cross-section, only prestressing steel placed near the surface shall be regarded as tensile steel.

8 Stress limit

8.1 General

In regard to stress generated in a cross-section of each member under a primary load and particular load equivalent to a primary load, the stress limits of a reinforced concrete member and prestressed concrete member shall be the values specified in 8.2 and 8.3, respectively.

The stress limit for incorporating the subsidiary load and a particular load equivalent to a subsidiary load shall be the values obtained by multiplying the stress limits specified in 8.2 and 8.3 by the extra coefficients given in Table 5.

The tensile stress limit of concrete in a prestressed concrete member for incorporating a subsidiary load and a particular load equivalent to a subsidiary load shall be the values specified in 8.3.4. The stress limit immediately after prestressing specified in 8.3.1 and 8.3.2 shall not be augmented.

NOTE 1 The stress limits specified in this document are equivalent to the safety (ultimate limit state) and serviceability (water leakage, etc. for serviceability limit state) verification.

NOTE 2 The extra coefficient specified in this document is to increase the stress limits in consideration with the probability of load combinations.

NOTE 3 Examples of stress limits are given in Annex D.

Table 5 — Combinations of loads and extra coefficients for stress limit

Combination of loads		Extra coefficient
a)	Primary load + particular load equivalent to primary load + effect of temperature (temperature difference)	[1,15]
b)	Primary load + particular load equivalent to primary load + wind load	[1,25]
c)	Primary load + particular load equivalent to primary load + effect of a Level 1 ground motion	[1,50]

8.2 Stress limit of reinforced concrete members

8.2.1 Stress limit of concrete

The compressive stress, shear stress and bond stress limits of concrete shall be determined to ensure performance requirements of a PC water tank.

The bearing stress limit of concrete shall be the value calculated by Formula (23).

$$\sigma'_{ba} = \left(0{,}25 + 0{,}05 \frac{A_c}{A_b}\right) f'_{ck} \tag{23}$$

$$\sigma'_{ba} \le 0{,}5 f'_{ck} \tag{24}$$

where

σ'_{ba} is the bearing stress limit of concrete;

A_c is the total area of the concrete surface;

A_b is the area of concrete subjected to bearing load;

f'_{ck} is the characteristic compressive strength of concrete.

8.2.2 Stress limit of reinforcement

The stress limit of reinforcement with a diameter of not more than 32 mm shall be determined taking into account the performance requirements of a PC water tank.

The stress limit shall be applied for members in contact with water in consideration of the fact that a PC water tank requires watertightness. However, those for general members should as a rule be applied to portions not in contact with water or effectively waterproofed portions.

When considering a combination of a primary load, particular load equivalent to a primary load and subsidiary load, the stress limit shall be the value obtained by multiplying the value for general members by the extra coefficient given in Table 5.

8.3 Stress limit of prestressed concrete members

8.3.1 Stress limit of concrete

The compressive, shear and diagonal tensile and bond stress limits of concrete shall be determined taking into account the performance requirements of a PC water tank.

The bearing stress limit shall be the value specified in 8.2.1. As to the backsides of anchorages, verification of bearing stress may be omitted by confirming that the compressive strength of concrete specified in 7.2.1 is attained at the time of applying prestress.

8.3.2 Tensile stress limit of prestressing steel

The tensile stress limit of prestressing steel shall be determined taking into account the performance requirements of a PC water tank.

8.3.3 Stress limit of reinforcement

The stress limit of reinforcement shall be as specified in 8.2.2.

8.3.4 Augmentation of tensile stress limit of concrete

The tensile stress limit of concrete considering a subsidiary load shall be determined with a combination of the flexural and axial tensile stress limits.

When considering a particular load equivalent to a subsidiary load, an adequate value should be established on a case-by-case basis in consideration of a PC water tank.

9 Verification of safety against earthquake

9.1 Principles of seismic design

9.1.1 General

A PC water tank shall be designed to ensure the performance that a structure should retain during an earthquake with respect to the combination of the ground motion level and the importance of the facility.

For seismic design, the construction characteristics of the structure and the ground characteristics of the area shall be taken into consideration to select a method suitable for these characteristics.

In general, the PC water tank is often adopted to prevent a secondary disaster during an earthquake and so that water storing function shall be ensured such that storing water can be used as emergency water. Therefore, the PC water tank shall be designed as high degree of importance.

9.1.2 Ground motion levels

For seismic design, Level 1 ground motion and Level 2 ground motion shall be considered as necessary. Level 1 ground motion represents an event with a probability of encountering once or twice during the life of the facility and Level 2 ground motion represents stronger ground motion with a lower probability of encountering.

NOTE Level 2 ground motion includes large-scale plate-boundary earthquakes occurring near land and inland near-field earthquakes. Though the probability of a waterworks facility encountering a ground motion of this level is generally low, its impact, once encountered, would be serious.

9.1.3 Levels of earthquake resistance

A PC water tank is required to retain the level of earthquake resistance specified in Table 6 as a standard against each ground motion level. The functions of the PC water tank are retainable even with minor damage to the tank.

Table 6 — Earthquake resistance level required for a PC water tank

Ground motion level	Earthquake resistance level
Level 1	No damage
Level 2	No severe impact on human life and fatal accidents. The functions of the PC water tank are retainable even with minor damage to the tank.

9.1.4 Effects of earthquake

For seismic design of a PC water tank, the following effects of an earthquake shall be considered:

— displacement and strain of the ground during an earthquake;

— inertia force derived from the deadweight of the structure, etc.;

— earth pressure during an earthquake;

— dynamic water pressure during an earthquake;

— sloshing;

— lateral flow of the ground due to liquefaction;

— strain of the ground in artificially altered sloping ground.

However, the earth pressure during an earthquake can be ignored because it is a subsidiary load in a PC water tank constructed on ground where this document is applied. When water colliding with the roof does not affect the roof, the effect of the sloshing can be ignored.

9.1.5 Seismic design procedure

Seismic design shall basically follow these procedures:

— select the construction site;

— carry out ground survey and soil exploration at the site;

— investigate the ground conditions;

— calculate the earthquake resistance;

— verify the earthquake resistance level.

9.2 Input earthquake motion

9.2.1 Seismic design method

Resistance to Level 1 ground motion shall be calculated basically by elastic analysis and as a rule by the seismic coefficient method. It is necessary to appropriately establish the design seismic coefficient depending on the ground type and the natural period of the structure.

Resistance to Level 2 ground motion shall be calculated as a rule by the seismic coefficient method. However, in the case of a PC water tank with a large scale and complicated shape, it is advisable to carry out dynamic analysis as well, as necessity arises, to verify safety in consideration of its characteristics (natural period).

The natural period of a PC water tank in a “tank full” condition may be determined by Formulae (25) and (26):

$$T = \frac{\pi H^2}{R}\sqrt{\frac{2q'}{3gE}\left\{1+12\left(\frac{R}{H}\right)^2\right\}} \tag{25}$$

$$q' = q_1 + \frac{q_0 R}{2t}\,\frac{\tanh\left(\frac{\sqrt{3}R}{H}\right)}{\frac{\sqrt{3}R}{H}} \tag{26}$$

where

t is the thickness of the tank wall;

R is the tank radius;

H is the total water depth of the tank;

g is the gravitational acceleration;

E is the elastic modulus;

q_1 is the unit weight of the wall;

q_0 is the unit weight of the liquid (water).

For a PC water tank having an irregular shape, the seismic coefficient method may not be capable of fully expressing its behaviour during an earthquake. In such a case, it is advisable to verify the results of the seismic design based on the seismic coefficient method by dynamic analysis in consideration of the characteristics (natural period) of the structure. During this process, nonlinear properties due to decrease of rigidity may be considered for modelling the structure. Analysis may also be conducted by the equivalent linear method.

A PC water tank to be built on ground generally has a simple shape of a cylinder and a relatively short natural period. For such a tank, design by the seismic coefficient method is sufficient without dynamic analysis.

9.2.2 Design seismic coefficients for the seismic coefficient method for Level 1 ground motion

The design horizontal seismic coefficient for Level 1 ground motion (K_{h1}) shall be determined as Formula (27):

$$K_{h1} = C_z K_{h01} \tag{27}$$

where

C_z is the correction factor by region (level of seismic hazard);

K_{h01} is the standard horizontal seismic coefficient at the position of the centroid of the structure.

The response of a waterworks facility to ground motion varies depending on the intensity, period characteristics and duration of the ground motion, ground type, type of the structure and type of the foundation. The standard horizontal seismic coefficients shall be based on the acceleration response spectrum curve for each ground type determined by analysing strong motion seismograms in each region. C_z and K_{h01} may be estimated referring to ISO 3010 or Annex B.

When considering the design vertical seismic coefficient (K_{v1}), the value shall be determined by $K_{v1} = K_{h1}/2$. For the seismic calculation of a normal structure, consideration of the effect of ground motion only in the horizontal direction is sufficient for constructing an earthquake-resistant structure. However, it may be desirable for certain types or parts of structures, such as anchorages to the foundations of tank structures having standpipes and the overhanging bottoms of elevated water tanks, to investigate the effects of vertical ground motion.

9.2.3 Design seismic coefficients for the seismic coefficient method for Level 2 ground motion

The design horizontal seismic coefficient for Level 2 ground motion (K_{h2}) shall be determined as Formula (28):

$$K_{h2} = C_s K_{h02} \tag{28}$$

where

C_s is the characteristic coefficient of the structure. This shall be appropriately established for the degrees of the response attenuation and plastic deformability of the structure. However, C_s = 0,45 may be assumed for a PC water tank of a general shape;

K_{h02} is the standard horizontal seismic coefficient at the position of the centroid of the structure.

The standard horizontal seismic coefficient for Level 2 ground motion at the position of the centroid of the structure as specified for each ground type is beyond the scope of this document. K_{h02} shall be estimated referring to ISO 3010 or Annex B.

The structure characteristic coefficient, C_s, should be determined by Formula (29):

$$C_s = D.\, D_{he} \tag{29}$$

where

D_{he} is the correction factor dependent on damping constant. This may be assumed to be 1,0 when the damping constant is 5 %;

D_η is the response reduction ratio due to plastic deformability determined by Formula (30).

$$D. = \frac{1}{\sqrt{2\left(\frac{Q_a}{Q_y}+1\right)\eta+1}} \tag{30}$$

where

Q_y is the yield load of member;

Q_a is the load at the time of allowable displacement;

η is the mean cumulative plastic magnification expressed by Formula (31).

$$\eta = \frac{\delta_0}{\delta_y} \tag{31}$$

where

δ_0 is the cumulative plastic deformation;

δ_y is the yield displacement.

NOTE With regard to a general shaped PC water tank with a capacity of 1 000 m^3 to 30 000 m^3, it is possible to determine D_η = 0,45, that is, C_s = 0,45.

When considering the design vertical seismic coefficient (K_{v2}), the value shall be determined by $K_{v2} = K_{h2}/2$.

9.2.4 Seismic input for design by dynamic analysis

When using dynamic analysis to verify for the safety, the following seismic waves shall be used: seismic waves conforming to the velocity response spectra of bedrock and the acceleration response spectra of surface ground or actual seismic waves observed near inland faults at site.

For defining the seismic wave, the requirements of ISO 3010 or Annex C may be used.

9.3 Verification of structural safety

9.3.1 Effects of earthquake

9.3.1.1 General

The effects of an earthquake to be considered shall include the following:

— inertia force derived from the deadweight of the structure, etc.;

— dynamic water pressure during an earthquake

The inertia force derived from deadweight, etc. and dynamic water pressure during an earthquake, which shall be specifically considered in the verification of seismic safety of a PC water tank are specified in detail.

9.3.1.2 Inertia force derived from deadweight, etc

The inertia force derived from the deadweight and imposed loads of the structure shall be determined by multiplying their respective weights by the design seismic coefficients specified in 9.2.2 and 9.2.3, giving sufficient consideration to the characteristics of the structure.

9.3.1.3 Dynamic water pressure during an earthquake

As dynamic water pressure during an earthquake, the following shall be considered:

— dynamic water pressure responding to short-period components of an earthquake (impulsive pressure);

— dynamic water pressure responding to relatively long-period components of an earthquake (convective pressure)

In addition to the above, pressure due to the elastic deformation of the tank and pressure due to rocking are conceivable as pressure acting on the tank during an earthquake. However, their effects on PC water tanks of a general shape are so small that these may be disregarded.

The Housner method and velocity potential method are representative methods of calculating dynamic water pressure during an earthquake. Other methods include the finite element method, boundary element method and transfer matrix method. Any of these methods should be used for calculating dynamic water pressure during an earthquake.

The contained water, when divided into solid that can be regarded as solid mass and solid oscillating mass that vibrates, can be replaced with a so-called spring-mass model. The weights of solid mass and solid oscillating mass of water and the impulsive and convective pressures are related as follows:

$$\alpha_s = \frac{g}{W_s} \int_0^H \int_0^{2\pi} P_{ws}(\xi) R \cos^2\theta \, d\theta \, d\xi \tag{32}$$

$$\alpha_r = \frac{g}{W_r} \int_0^H \int_0^{2\pi} P_{wr}(\xi) R \cos^2\theta \, d\theta \, \xi \tag{33}$$

where

α_s, α_r	are the response acceleration of solid oscillating and solid mass points, respectively, of the s-th mode;
W_s, W_r	are the weights of solid oscillating mass of water and solid mass of water, respectively, of the s-th mode;
$P_{ws}(\xi)$, $P_{wr}(\xi)$	are the maximum convective pressure and impulsive pressure, respectively, on the tank wall;
ξ, θ	are the distance from the water surface and angle of earthquake to the horizontal plane;
H	is the water depth.

Table 7 compares the formulae used in the Housner method and the velocity potential method.

Table 7 — Formulae for calculating dynamic water pressure

	Housner method	Velocity potential method
$\omega_s{}^2$	$1{,}837 \frac{g}{R} \tanh\left(1{,}837 \frac{H}{R}\right)$	$k_s \frac{g}{R} \tanh\left(k_s \frac{H}{R}\right)$
W_s	$0{,}3189 \frac{R}{H} \tanh\left(1{,}837 \frac{H}{R}\right) W$	$\frac{2R}{(k_s^2 - 1) k_s H} \tanh\left(k_s \frac{H}{R}\right) W$
W_r	$\frac{\tanh\left(\sqrt{3} \frac{R}{H}\right)}{\sqrt{3} \frac{R}{H}} W$	$W - \sum_{s=1}^{\infty} W_s$
P_r	$K_h W_r$	$K_h W_r$
P_s	$\omega_s S_v \frac{W_s}{g}$	$\omega_s S_v \frac{W_s}{g}$
h_{sE}	$\left[1 - \frac{\cosh\left(1{,}837 \frac{H}{R}\right) - 1}{1{,}837 \frac{H}{R} \sinh\left(1{,}837 \frac{H}{R}\right)}\right] H$	$H - \frac{R}{k_s} \tanh\left(\frac{k_s}{2} \frac{H}{R}\right)$
h_{rE}	$\frac{3}{8} H$	$\frac{1}{W_r}\left(\frac{WH}{2} - \sum_{s=1}^{\infty} W_s h_{sE}\right)$

Table 7 *(continued)*

	Housner method	Velocity potential method
h_{sI}	$\left[1-\frac{\cosh\left(1{,}837\frac{H}{R}\right)-2{,}013}{1{,}837\frac{H}{R}\sinh\left(1{,}837\frac{H}{R}\right)}\right]H$	$H-\frac{R}{k_s}\left[\coth\left(k_s\frac{H}{R}\right)-2\mathrm{cosech}\left(k_s\frac{H}{R}\right)\right]$
h_{rI}	$\frac{1}{8}\left[\frac{4}{\frac{\tanh\left(\sqrt{3}\frac{R}{H}\right)}{\sqrt{3}\frac{R}{H}}}-1\right]H$	$\frac{1}{W_r}\left[\frac{W}{2}\left(H+\frac{R^2}{2H}\right)-\sum_{s=1}^{\infty}W_s h_{sI}\right]$
$P_{wr}(\xi)$	$\sqrt{3}\rho K_h H\left[\frac{\xi}{H}-\frac{1}{2}\left(\frac{\xi}{H}\right)^2\right]\tanh\left(\sqrt{3}\frac{R}{H}\right)$	$\rho K_h R\left[1-2\sum_{s=1}^{\infty}\frac{1}{k_s^2-1}\frac{\cosh\left(k_s\frac{H-\xi}{R}\right)}{\cosh\left(k_s\frac{H}{R}\right)}\right]$
$P_{br}(r)$	$\frac{\sqrt{3}}{2}\rho K_h H\frac{\sinh\left(\sqrt{3}\frac{r}{H}\right)}{\cosh\left(\sqrt{3}\frac{R}{H}\right)}$	$\rho K_h\left[r-2R\sum_{s=1}^{\infty}\frac{1}{k_s^2-1}\frac{J_1\left(k_s\frac{r}{R}\right)}{J_1(k_s)\cosh\left(k_s\frac{H}{R}\right)}\right]$
$P_{ws}(\xi)$	$0{,}340\,2\rho\left(\frac{R}{g}\right)^2\omega_s^3 S_v\frac{\cosh\left(1{,}837\frac{H-\xi}{R}\right)}{\sinh\left(1{,}837\frac{H}{R}\right)}$	$2\rho\frac{R}{g}\omega_s S_v\frac{1}{k_s^2-1}\frac{\cosh\left(k_s\frac{H-\xi}{R}\right)}{\cosh\left(k_s\frac{H}{R}\right)}$
$P_{bs}(r)$	$0{,}510\,4\rho\left(\frac{R}{g}\right)^2\omega_s^3 S_v\left[\left(\frac{r}{R}\right)-\frac{1}{3}\left(\frac{r}{R}\right)^3\right]\frac{1}{\sinh\left(1{,}837\frac{H}{R}\right)}$	$2\rho\frac{R}{g}\omega_s S_v\frac{1}{k_s^2-1}\frac{J_1\left(k_s\frac{r}{R}\right)}{J_1(k_s)\cosh\left(k_s\frac{H}{R}\right)}$

where

W is the total weight of contained water;

ω_s is the natural circular frequency of the s-th mode;

W_s is the weight of solid oscillating mass point of the s-th mode;

W_r is the weight of solid mass point;

P_r is the horizontal force acting on solid mass point;

P_s is the horizontal force acting on solid oscillating mass point of the s-th mode;

h_{sE}, h_{rE} are the distances from the tank bottom to the solid oscillating mass point and solid mass point, respectively, when the dynamic water pressure acting on the base slab is disregarded;

h_{sI}, h_{rI} are the distances from the tank bottom to the solid oscillating mass point and solid mass point, respectively, when the dynamic water pressure on the base slab is considered;

$P_{wr}(\xi)$, $P_{ws}(\xi)$ are the maximum impulsive pressure and convective pressure, respectively, acting on the tank wall;

$P_{br}(r)$, $P_{bs}(r)$ are the maximum impulsive pressure and convective pressure, respectively, acting on the base slab;

subscript s is the mode number of solid oscillating mass point (s = 1 in the Housner method); s = 1 is also normally sufficient in the velocity potential method;

g, ρ are the gravitational acceleration and unit weight of water, respectively;

R, H are the tank radius and water depth, respectively;

k_s is the positive root that satisfies $\frac{d}{dk_s}\left[J_1\left(k_s\right)\right]=0$ (see Table 8);

S_v is the velocity response spectrum;

K_h is the design horizontal seismic coefficient (K_{h1} and K_{h2} for Level 1 and Level 2 ground motions, respectively);

J_1 is the vessel function (refer to Table 9);

ξ, r are the vertical distance from water surface and horizontal distance from the centre of the tank, respectively.

Table 8 — k_s

s	k_s
1	1,841
2	5,331
3	8,536
4	11,706
5	14,863

Table 9 — Vessel function, J_1

	J_1 (k_s r/R)										
r/R	**0,0**	**0,1**	**0,2**	**0,3**	**0,4**	**0,5**	**0,6**	**0,7**	**0,8**	**0,9**	**1,0**
s = 1	0,0	0,091 66	0,181 0	0,265 8	0,343 8	0,413 2	0,472 2	0,519 5	0,554 0	0,574 9	0,581 9
= 2	0,0	0,257 2	0,460 9	0,569 8	0,564 7	0,452 0	0,261 9	0,040 75	−0,160 5	−0,298 1	−0,346 1
= 3	0,0	0,389 1	0,578 2	0,481 5	0,173 2	−0,161 5	−0,338 7	−0,281 5	−0,056 49	0,177 6	0,273 3
= 4	0,0	0,490 6	0,532 2	0,132 4	−0,275 3	−0,302 9	0,002 40	0,257 5	0,188 4	−0,086 90	−0,233 3
= 5	0,0	0,556 0	0,349 2	−0,219 8	−0,287 1	0,117 9	0,254 2	−0,056 45	−0,229 4	0,011 55	0,207 0

Impulsive pressure is never simultaneous with convective pressure. Impulsive pressure is greater than convective pressure on general PC water tanks. Investigation of the effect of convective pressure may therefore be omitted for a tank with a small diameter causing small sloshing. Also, these can be combined by determining the square root of the sum of squares.

When approximating the distribution of dynamic water pressure during an earthquake by a simple form, the shape should envelope the object dynamic water pressure distribution to the extent that it is possible. Figure 9 illustrates a typical example of approximated dynamic water pressure distribution. The pressures at the upper and lower edges should be determined by Formulae (34) and (35):

$$P_{\mathrm{l}} = \sqrt{P_{0\mathrm{l}}{}^{2} + P_{1\mathrm{l}}{}^{2}} \tag{34}$$

$$P_{\mathrm{u}} = \frac{P_{1\mathrm{u}} + P_{\mathrm{l}}}{2} \tag{35}$$

where

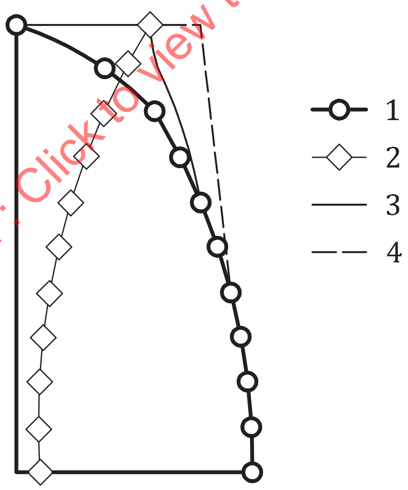
P_{u} is the dynamic water pressure at upper edge;

P_{l} is the dynamic water pressure at lower edge;

$P_{0\mathrm{l}}$ is the impulsive pressure at lower edge;

$P_{1\mathrm{u}}$ is the convective pressure at upper edge;

$P_{1\mathrm{l}}$ is the convective pressure at lower edge.



Key

1 impulsive pressure

2 convective pressure

3 root mean square combination

4 trapezoidal rule

Figure 9 — Approximation of dynamic water pressure distribution

9.3.2 Combination of loads

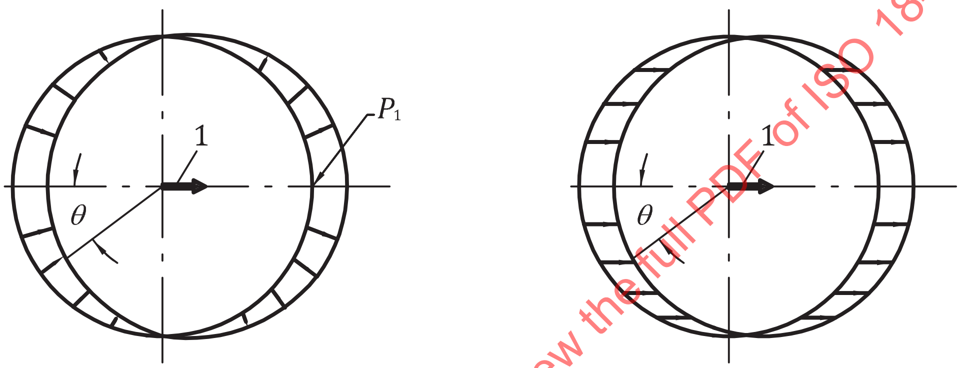
As to the combination of loads for verifying the degree of safety, only the “tank full” condition shall be considered because it is not likely the tank is kept empty for long time.

As to the combination of loads on a filled PC water tank, the imposed loads, the effects of creep and drying shrinkage of concrete, temperature and wind loads can be ignored from the list of loads given in Table 1. However, loads that strongly affect the tank wall shall be considered as necessary.

9.3.3 Calculation of member forces

Member forces induced by Level 1 and Level 2 ground motions shall be calculated by assuming the tank body as a thin-wall shell structure in consideration of the characteristics of the loading action during an earthquake.

Dynamic water pressure and the inertia force of the structural framing that act during an earthquake are distributed non-axisymmetrically in the circumferential direction as shown in Figure 10. It is, therefore, convenient to analyse the loads, displacements and member forces by expanding them in a Fourier series in the circumferential direction and using axisymmetric shell elements. However, since it is difficult to determine them in an analytical manner, the analysis may generally be carried out using the axisymmetric thin-wall shell finite element method, which can be expanded in a Fourier series.



$X = 0$

$Y = 0$

$Z = P_1 \cdot \cos\theta$

where P_1 is the dynamic water pressure at $\theta = 180°$.

a) Dynamic water pressure

$X = 0$

$Y = K_h \cdot \rho_c \cdot t \cdot \sin\theta$

$Z = K_h \cdot \rho_c \cdot t \cdot \cos\theta$

where

K_h is the design horizontal seismic coefficient;

ρ_c is the unit weight of concrete (wall);

t is the wall thickness.

b) Inertia force

Key

1 acting direction of earthquake force

Figure 10 — Load distribution during an earthquake

In the case of a general PC water tank, the displacement and member force of the tank wall may be approximately determined by the following method.

a) Calculation of circumferential axial force and vertical bending moment

1) Dominant formula of cylindrical shell subjected to non-axisymmetric loads

The dominant formula of a cylindrical shell on which non-axisymmetric loads act is approximately obtained using Formula (36):

$$\frac{d^4 w_{x0}}{dx^4} + \frac{12\left(1-\upsilon^2\right)}{\left(Rt\right)^2} w_{x0} = \frac{1}{K}\left[\frac{d^2 P_0(x)}{dx^2} - \frac{2}{R^2} P_0(x) - q_0(x) + C_1 x + C_2\right] \tag{36}$$

where

w_{x0} is the radial displacement of the tank wall at the point of maximum dynamic water pressure (θ = 0°);

K is the flexural stiffness.

$P_0(x)$ $= \int_0^{H-x} (H-x-\xi) p_0(\xi) d\xi$

$\frac{dP_0(x)}{dx}$ $= -\int_0^{H-x} p_0(\xi) d\xi$

$\frac{d^2 P_0(x)}{dx^2}$ $= p_0(x)$

$p_0(x)$ is the radial component of the load;

$q_0(x)$ is the circumferential component of the load;

C_1 $= -\frac{\upsilon}{R^2}\frac{dP_0(0)}{dx}$

C_2 $= \frac{2+\upsilon}{R^2} P_0(0)$

x is the distance from the lower edge of the tank wall;

ξ is the distance from the upper edge of the tank wall;

R is the tank radius;

H is the total water depth of the tank;

ν is the Poisson's ratio.

The left-hand member of Formula (36) has the same form as the dominant formula of a cylindrical shell subjected to axisymmetric loads. For this reason, the member force on a cylindrical shell subjected to non-axisymmetric loads can be determined by conversion shown below using displacements and member forces determined under loads assuming that the equivalent load in the right-hand member is axisymmetric.

2) Equivalent loads

i) Impulsive pressure (velocity potential method)

$$P_0(x) = \rho K_h R \left\langle \frac{(H-x)^2}{2} - 2\sum_{s=1}^{\infty} \frac{1}{k_s^2 - 1} \left\{ \left(\frac{R}{k_s} \right)(H-x) \tanh\left(k_s \frac{H}{R} \right) + \left(\frac{R}{k_s} \right)^2 \left[\frac{\cosh\left(k_s \frac{x}{R} \right)}{\cosh\left(k_s \frac{H}{R} \right)} - 1 \right] \right\} \right\rangle$$

$$\frac{d^2 P_0(x)}{dx^2} = p_0(x) = \rho K_h R \left[1 - 2\sum_{s=1}^{\infty} \frac{1}{k_s^2 - 1} \frac{\cosh\left(k_s \frac{x}{R} \right)}{\cosh\left(k_s \frac{H}{R} \right)} \right]$$

$$C_1 = \frac{\upsilon \rho K_h}{R} \left\{ H + 2\sum_{s=1}^{\infty} \frac{1}{k_s^2 - 1} \left(\frac{R}{k_s} \right) \left[-\tanh\left(k_s \frac{H}{R} \right) \right] \right\} \tag{37}$$

$$C_2 = \frac{(2+\upsilon) \rho K_h}{R} \left\langle \frac{H^2}{2} - 2\sum_{s=1}^{\infty} \frac{1}{k_s^2 - 1} \left\{ \left(\frac{R}{k_s} \right) H \tanh\left(k_s \frac{H}{R} \right) + \left(\frac{R}{k_s} \right)^2 \left[\frac{1}{\cosh\left(k_s \frac{H}{R} \right)} - 1 \right] \right\} \right\rangle$$

$$q_0(x) = 0$$

ii) Convective pressure (velocity potential method)

$$P_0(x) = \frac{2\rho R \omega_s S_v}{g} \frac{1}{k_s^2 - 1} \left\{ \left(\frac{R}{k_s} \right)(H-x) \tanh\left(k_s \frac{H}{R} \right) + \left(\frac{R}{k_s} \right)^2 \left[\frac{\cosh\left(k_s \frac{x}{R} \right)}{\cosh\left(k_s \frac{H}{R} \right)} - 1 \right] \right\}$$

$$\frac{d^2 P_0(x)}{dx^2} = \frac{2\rho R \omega_s S_v}{g} \frac{1}{k_s^2 - 1} \frac{\cosh\left(k_s \frac{x}{R} \right)}{\cosh\left(k_s \frac{H}{R} \right)}$$

$$C_1 = \frac{2\upsilon \rho \omega_s S_v}{g} \frac{1}{k_s^2 - 1} \left(\frac{R}{k_s} \right) \tanh\left(k_s \frac{H}{R} \right) \tag{38}$$

$$C_2 = \frac{2+\upsilon}{R} \frac{2\rho \omega_s S_v}{g} \frac{1}{k_s^2 - 1} \left\{ \left(\frac{R}{k_s} \right) H \tanh\left(k_s \frac{H}{R} \right) + \left(\frac{R}{k_s} \right)^2 \left[\frac{1}{\cosh\left(k_s \frac{H}{R} \right)} - 1 \right] \right\}$$

$$q_0(x) = 0$$

iii) Dynamic water pressure approximated to trapezoidal load

$$P_0(x)=\frac{\alpha_1}{3}(H-x)^3+\frac{\alpha_1 x+\beta_1}{2}(H-x)^2$$

$$\frac{d^2P_0(x)}{dx^2}=\alpha_1 x+\beta_1$$

$$C_1=\frac{\upsilon}{R^2}H\left(\frac{\alpha_1}{2}H+\beta_1\right) \tag{39}$$

$$C_2=\frac{2+\upsilon}{R^2}H^2\left(\frac{\alpha_1}{3}H+\frac{\beta_1}{2}\right)$$

$$q_0(x)=0$$

where

α_1 is the load gradient as defined in Figure 11 (The gradient shown in the figure is assumed to be negative.);

β_1 is the load at lower edge of the tank wall.

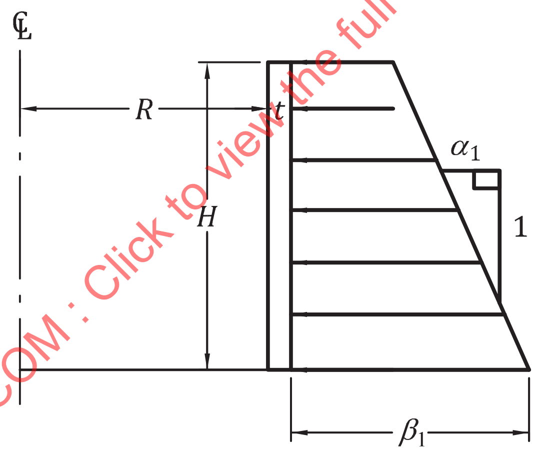


Figure 11 — α_1 and β_1

iv) Inertia force of the structural framing

$$P_0(x)=q_0(x)(H-x)^2$$

$$\frac{d^2P_0(x)}{dx^2}=2q_0(x)$$

$$C_1=\frac{2\upsilon q_0(x)H}{R^2} \tag{40}$$

$$C_2=\frac{2+\upsilon}{R^2}q_0(x)H^2$$

$$q_0(x)=K_h\rho_c t$$

where

K_h is the design horizontal seismic coefficient (K_{h1} and K_{h2} for Level 1 and Level 2 ground motions, respectively);

ρ_c is the unit weight of concrete;

t is the tank wall thickness.

3) Conversion of member force

The circumferential axial force due to non-axisymmetric loads, $N_{\phi 0}$, and the vertical bending moment, M_{x0}, are determined by the following Formulae (41) and (42):

$$N_{\varphi 0} = -\frac{Et}{R} w_{x0} - \frac{2-\upsilon}{R} \overline{M_x} - \frac{2}{R} P_0(x) + R(C_1 x + C_2) \tag{41}$$

$$M_{x0} = \overline{M_x} - \frac{1}{12}\left(\frac{t}{R}\right)^2 P_0(x) \tag{42}$$

where

$\overline{M_x}$ is the vertical bending moment obtained by axisymmetric analysis under equivalent loads;

w_{x0} is the radial displacement obtained by axisymmetric analysis under equivalent loads (the same as radial displacement under non-axisymmetric loads).

The values obtained by multiplying the determined member force by −1,0 and +1,0 are taken as the member forces at θ = 180° and θ = 0°, respectively.

When conducting axisymmetric analysis under equivalent loads, displacement and member force of the tank wall may be determined by using the formulae for displacement and member force for the case where arbitrary trapezoidal loads partially act on the tank wall. When equivalent loads are approximated to trapezoidal loads, the trapezoidal loads shall be approximated so that they would envelope the equivalent loads.

$$w_\xi = C_1 m_4(\beta\xi) + \frac{C_2}{2\beta}\left[m_2(\beta\xi) + m_3(\beta\xi)\right] + G_1(\xi) \tag{43}$$

$$N_\varphi = -\frac{Et}{R} w_\xi \tag{44}$$

$$M_\xi = -K\left\{C_1\left(-2\beta^2\right)m_1(\beta\xi) + C_2\beta\left[-m_2(\beta\xi) + m_3(\beta\xi)\right] + G_3(\xi)\right\} \tag{45}$$

where

K is the flexural stiffness;

β is the characteristic value of the tank wall;

ξ is the distance from the upper edge of the tank wall.

$$C_1 = \frac{-k(i_2,2)g(i_1)+k(i_1,2)g(i_2)}{k(i_1,1)k(i_2,2)-k(i_1,2)k(i_2,1)}$$

$$C_2 = \frac{-k(i_2,1)g(i_1)-k(i_1,1)g(i_2)}{k(i_1,1)k(i_2,2)-k(i_1,2)k(i_2,1)}$$

i_1, i_2 see Table 10.

Table 10 — i_1 and i_2

	i_1	i_2
Unrestrained support	3	4
Hinged support	1	3
Fixed support	1	2

$$k(1,1)=k(2,2)=m_4(\beta H)$$

$$k(1,2)=\frac{1}{2\beta}\{m_2(\beta H)+m_3(\beta H)\}$$

$$k(2,1)=k(3,2)=\beta\{-m_2(\beta H)+m_3(\beta H)\}$$

$$k(3,1)=k(4,2)=-2\beta^2 m_1(\beta H)$$

$$k(4,1)=-2\beta^3\{m_2(\beta H)+m_3(\beta H)\}$$

$$g(1)=\frac{w_1}{K}\frac{1}{4\beta^4}\left[-m_4\{\beta(H-l_1)\}+m_4\{\beta(H-l_2)\}\right]+$$

$$\frac{w_2}{K}\frac{1}{l_2-l_1}\left[\frac{l_2-l_1}{4\beta^4}-\frac{1}{8\beta^5}\left[m_2\{\beta(H-l_1)\}\right]+m_3\{\beta(H-l_1)\}-m_2\{\beta(H-l_2)\}-m_3\{\beta(H-l_2)\}\right]-$$

$$\frac{w_2}{K}\frac{1}{4\beta^4}\left[1-m_4\{\beta(H-l_2)\}\right]$$

$$g(2)=\frac{w_1}{K}\frac{1}{4\beta^3}\left[m_2\{\beta(H-l_1)\}-m_3\{\beta(H-l_1)\}-m_2\{\beta(H-l_2)\}+m_3\{\beta(H-l_2)\}\right]+$$

$$\frac{w_2}{K}\frac{1}{l_2-l_1}\left(-\frac{1}{4\beta^4}\right)\left[m_4\{\beta(H-l_1)\}-m_4\{\beta(H-l_2)\}-\frac{w_2}{K}\frac{1}{4\beta^3}\left[m_2\{\beta(H-l_2)\}-m_3\{\beta(H-l_2)\}\right]\right]$$

$$g(3)=\frac{w_1}{K}\frac{1}{2\beta^2}\left[m_1\{\beta(H-l_1)\}-m_1\{\beta(H-l_2)\}\right]+$$

$$\frac{w_2}{K}\frac{1}{l_2-l_1}\left(-\frac{1}{4\beta^3}\right)\left[-m_2\{\beta(H-l_2)\}+m_3\{\beta(H-l_1)\}+m_2\{\beta(H-l_2)\}-\right.$$

$$\left.m_3\{\beta(H-l_2)\}\right]-\frac{w_2}{K}\frac{1}{2\beta^2}m_1\{\beta(H-l_2)\}$$

$$g(4)=\frac{w_1}{K}\frac{1}{2\beta}\left[m_2\{\beta(H-l_1)\}+m_3\{\beta(H-l_1)\}-m_2\{\beta(H-l_2)\}-m_3\{\beta(H-l_2)\}\right]+$$

$$\frac{w_2}{K}\frac{1}{l_2-l_1}\left(-\frac{1}{2\beta^2}\right)\left[-m_1\{\beta(H-l_1)\}+m_1\{\beta(H-l_2)\}\right]-\frac{w_2}{K}\frac{1}{2\beta}\left[m_2\{\beta(H-l_2)\}+m_3\{\beta(H-l_2)\}\right]$$

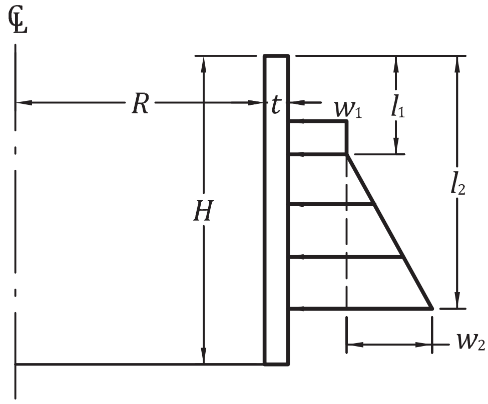


Figure 12 — Symbols

Definitions of the symbols are given in Figure 12.

$m_1(\beta\xi)=\sin(\beta\xi)\sinh(\beta\xi)$

$m_2(\beta\xi)=\sin(\beta\xi)\cosh(\beta\xi)$

$m_3(\beta\xi)=\cos(\beta\xi)\sinh(\beta\xi)$

$m_4(\beta\xi)=\cos(\beta\xi)\cosh(\beta\xi)$

$0\le\xi\le l_1$

$G_1(\xi)=G_3(\xi)=0$

$l_1\le\xi\le l_2$

$$G_1(\xi)=\frac{w_1}{K}\frac{1}{4\beta^4}\left[1-m_4\{\beta(\xi-l_1)\}\right]+\frac{w_2}{K}\frac{1}{l_2-l_1}\left\langle\frac{\xi-l_1}{4\beta^4}-\frac{1}{8\beta^5}\left[m_2\{\beta(\xi-l_1)\}+m_3\{\beta(\xi-l_1)\}\right]\right\rangle$$

$$G_3(\xi)=\frac{w_1}{K}\frac{1}{2\beta^2}m_1\{\beta(\xi-l_1)\}+\frac{w_2}{K}\frac{1}{l_2-l_1}\frac{1}{4\beta^3}\left[m_2\{\beta(\xi-l_1)\}-m_3\{\beta(\xi-l_1)\}\right]$$

$l_2\le\xi\le H$

$$G_1(\xi)=\frac{w_1}{K}\frac{1}{4\beta^4}\left[-m_4\{\beta(\xi-l_1)\}+m_4\{\beta(\xi-l_2)\}\right]+$$

$$\frac{w_2}{K}\frac{1}{l_2-l_1}\left\langle\frac{l_2-l_1}{4\beta^4}-\frac{1}{8\beta^5}\left[m_2\{\beta(\xi-l_1)\}+m_3\{\beta(\xi-l_1)\}-m_2\{\beta(\xi-l_2)\}-m_3\{\beta(\xi-l_2)\}\right]\right\rangle-$$

$$\frac{w_2}{K}\frac{1}{4\beta^4}\left[1-m_4\{\beta(\xi-l_2)\}\right]$$

$$G_3(\xi)=\frac{w_1}{K}\frac{1}{2\beta^2}\left[m_1\{\beta(\xi-\ell_1)\}-m_1\{\beta(\xi-\ell_2)\}\right]+$$

$$\frac{w_2}{K}\frac{1}{\ell_2-\ell_1}\left(-\frac{1}{4\beta^3}\right)\left[-m_2\{\beta(\xi-\ell_1)\}+m_3\{\beta(\xi-\ell_1)\}+m_2\{\beta(\xi-\ell_2)\}-m_3\{\beta(\xi-\ell_2)\}\right]-$$

$$\frac{w_2}{K}\frac{1}{2\beta^2}m_1\{\beta(\xi-\ell_2)\}$$

where

H is the tank wall height;

w_1, w_2 are the strengths of uniformly distributed loads and triangularly distributed loads;

l_1, l_2 are the distances from the upper edge of the tank wall to the starting and end points of the action of distributed loads.

M_{x0} and $N_{\phi0}$ of the tank wall due to the inertia force of the dome may be disregarded in the cross-sections under analysis at the lower edge and mid-height of the tank wall.

In regard to the radial and circumferential bending moments of the base slab, the member force may be determined by assuming that the action of the inertia force and maximum dynamic water pressure during an earthquake (θ = 180°) is axisymmetric. The values obtained by multiplying the determined member force by ±1,0 are taken as the member forces at θ = 180° and 0°.

b) Calculation of vertical axial force

The vertical axial force due to loads during an earthquake, N_{x0}, is determined by Formula (46):

$$N_{x0}(x)=\frac{M_{0T}(x)t}{Z}=\frac{M_{0T}(x)}{\pi R^2} \tag{46}$$

where

$N_{x0}(x)$ is the vertical axial force at distance x from the lower edge of the tank wall;

$M_{0T}(x)$ is the moment at distance x from the lower edge of the tank wall;

Z is the section modulus for the distance from the neutral axis to the compression edge when the tank is assumed to be a thin-wall ring;

t is the tank wall thickness.

The values obtained by multiplying the determined member force by ±1,0 are taken as the member forces at θ = 180° and 0°.

$M_{0T}(x)$, which results from the dynamic water pressure, convective pressure, dynamic water pressure approximated to trapezoidal load, inertia force of the structural framing, inertia force of the dome and other partial weight inertia forces, may be obtained from Formulae (47) to (51):

1) $M_{0T}(x)$ due to dynamic water pressure

$$M_{0T}(x)=\int_0^{H-x}(H-x-\xi)p_0(\xi)d\xi\cdot 2\int_{-\pi/2}^{\pi/2}R\cos^2\theta d\theta$$

$$=\rho K_h R\left\langle \frac{(H-x)^2}{2}-2\sum_{s=t}^{\infty}\frac{1}{k_s^2-1}\left\{\left(\frac{R}{k_s}\right)(H-x)\tanh\left(k_s\frac{H}{R}\right)\right\}+\left(\frac{R}{k_s}\right)^2\left[\frac{\cosh\left(k_s\frac{x}{R}\right)}{\cosh\left(k_s\frac{H}{R}\right)}-1\right]\right\rangle \pi R \tag{47}$$

2) $M_{0T}(x)$ due to convective pressure

$$M_{0T}(x)=\int_0^{H-x}(H-x-\xi)p_0(\xi)d\xi\cdot 2\int_{-\pi/2}^{\pi/2}R\cos^2\theta d\theta$$

$$=\frac{2\rho R\omega_s S_v}{g}\frac{1}{k_s^2-1}\left\{\left(\frac{R}{k_s}\right)(H-x)\tanh\left(k_s\frac{H}{R}\right)+\left(\frac{R}{k_s}\right)^2\left(\frac{\cosh\left(k_s\frac{X}{R}\right)}{\cosh\left(k_s\frac{H}{R}\right)}-1\right)\right\}\pi R \tag{48}$$

3) $M_{0T}(x)$ due to dynamic pressure approximated to trapezoidal load

$$M_{0T}(x)=\int_0^{H-x}(H-x-\xi)p_0(\xi)d\xi\cdot 2\int_{-\pi/2}^{\pi/2}R\cos^2\theta d\theta=\left(\frac{\alpha_1}{3}(H-x)^3+\frac{\alpha_1 x+\beta_1}{2}(H-x)^2\right)\pi R \tag{49}$$

where

α_1 is the pressure gradient;

β_1 is the pressure at the lower edge of the tank wall.

4) $M_{0T}(x)$ due to inertia force of the structural framing

$$M_{0T}(x)=\int_0^{H-x}(H-x-\xi)K_h\rho_c t d\xi\cdot 2\int_{-\pi/2}^{\pi/2}Rd\theta=\frac{(H-x)^2}{2}K_h\rho_c t\cdot 2\pi R \tag{50}$$

where

K_h is the design horizontal seismic coefficient (K_{h1} and K_{h2} for Level 1 and Level 2 ground motions, respectively);

ρ_c is the unit weight of concrete;

t is the tank wall thickness.

5) $M_{0T}(x)$ due to inertia force of the dome or other partial weight inertia forces

$$M_{0T}(x)=(H_G-x)WK_h \tag{51}$$

where

K_h is the design horizontal seismic coefficient (K_{h1} and K_{h2} for Level 1 and Level 2 ground motions, respectively);

H_G is the distance from the lower edge of the tank wall to the point of action of the dome inertia force or other partial weight inertia forces;

W is the dome weight or other partial weights.

9.3.4 Safety verification

Safety for Level 1 ground motion shall be verified by confirming that the PC water tank attains the level of earthquake resistance to be retained during an earthquake specified in 9.1.3.

Attainment of the level of earthquake resistance of "no damage" shall be verified specifically by calculating the member force due to the combination of loads during an earthquake that would act on the member and confirming that the resulting stress does not exceed the value specified in Clause 8. When using precast members having joints across which the reinforcement is not continuous, no tensile stress shall be generated in the joints.

When vertical prestress is applied to a PC water tank of a general shape, verification at θ = 0° and 180° is sufficient for the tank wall. Though in-plane shear forces act at θ = 90°, it is unthinkable that a diagonal stress due to in-plane shear forces would exceed its limit. Investigation at θ = 90° may therefore be omitted. However, investigation shall be carried out where in-plane shear force is of concern for the tank wall.

Also, it is unthinkable that the diagonal tensile stress due to out-of-plane sharing forces during an earthquake would exceed the regular diagonal tensile stress due to out-of-plane shear forces in a "tank empty" condition. For this reason, investigation of out-of-plane shear forces during an earthquake may be omitted.

Safety for Level 2 ground motions shall be verified by confirming that the PC water tank attains the level of earthquake resistance to be retained during an earthquake specified in 9.1.3.

Attainment of the level of earthquake resistance of "no serious adverse effect on human life" may be confirmed by confirming that the PC tank will not collapse. Attainment of the level of earthquake resistance of "the functions of the PC tank are retainable" should be confirmed by confirming that the watertightness of the tank is secured after Level 2 ground motion.

In regard to a PC water tank of a general shape, attainment of the above-mentioned two levels of earthquake resistance of the tank wall should be confirmed with the following:

— Circumferential

  Attainment of the level of earthquake resistance of "no severe adverse effect on human life" shall be confirmed by ensuring that the design circumferential tensile bearing capacity is greater than the design circumferential tensile force.

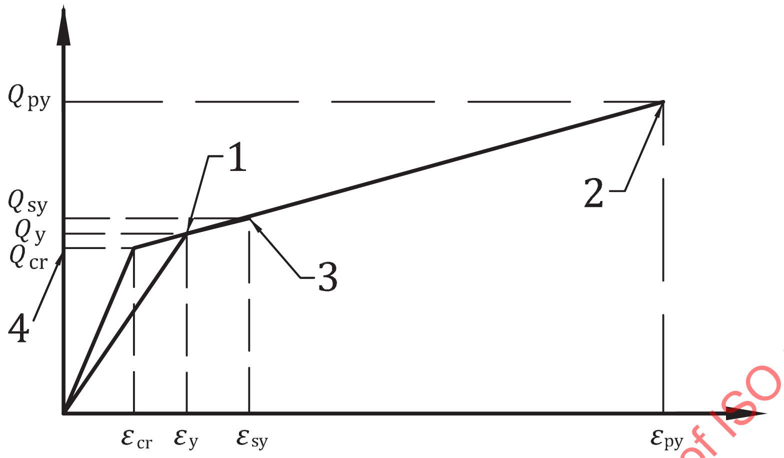
  Attainment of the level of earthquake resistance of "the functions of the PC water tank are retainable" shall be confirmed by ensuring that the response circumferential strain is smaller than the circumferential strain limit.

— Vertical

  Attainment of the levels of earthquake resistance of "no severe adverse effect on human life" and "the functions of the PC water tank are retainable" shall be confirmed by ensuring that the design vertical flexural bearing capacity is greater than the design vertical flexural moment.

NOTE Investigation by dynamic linear analysis and static nonlinear analysis on PC water tanks of a general shape has revealed that the failure mode is the axial tensile failure in the circumferential direction due to dynamic water pressure.

Specifically, verification shall be carried out by the following procedure. Circumferential axial force vs strain curve is indicated in Figure 13.



Key

1 virtual yielding point

2 yielding point of prestressing steel

3 yielding point of steel reinforcement

4 cracking load

Figure 13 — Circumferential axial force-strain curve

— Investigation of circumferential axial tensile force

$$\frac{Q_a}{Q_{he}} \geq \gamma_i \tag{52}$$

where

Q_a is the design circumferential tensile bearing capacity of member $\left[= \frac{A_p \left(f_{py} - \sigma_{pe} \right)}{\gamma_m \gamma_b} \right]$;

A_p is the cross-sectional area of prestressing steel;

f_{py} is the yield strength of prestressing steel;

σ_{pe} is the effective stress of prestressing steel;

Q_{he} is the circumferential axial tensile force determined by elastic analysis under an earthquake load calculated based on design horizontal seismic coefficient, K_{h2};

γ_i is the structure factor (= 1,0);

γ_m is the material factor (= 1,0);

γ_b is the member factor (may be set at 1,0 because this investigation deals with the axial tensile force of prestressing steel).

Investigation for θ = 180° is sufficient for a PC water tank of a general shape.

— Investigation of circumferential response strain

$$\varepsilon_r \le \varepsilon_{ra} \tag{53}$$

where

ε_r is the circumferential response strain $\left[= \varepsilon_y \left(1 + \eta_r\right) \right]$;

ε_y is the virtual yield strain of member $\left(= \dfrac{\varepsilon_{cr} + \varepsilon_{sy}}{2} \right)$;

ε_{cr} is the strain at crack onset ($= f_t/E_c$);

f_t is the tensile strength of concrete;

E_c is the elastic modulus of concrete;

ε_{sy} is the strain at reinforcement yielding ($= f_{sy}/E_s$);

f_{sy} is the yield strength of steel reinforcement;

E_s is the elastic modulus of steel reinforcement;

η_r is the response mean cumulative plastic deformation magnification $\left(= -\dfrac{1}{r_h} + \dfrac{1}{r_h} \sqrt{1 - \dfrac{r_h}{2} \left\{ 1 - \left(\dfrac{Q_e}{Q_y} \right)^2 \right\}} \right)$;

r_h is the ratio of secondary stiffness of member after yielding to yield stiffness $\left(= \dfrac{\dfrac{Q_{py} - Q_y}{\varepsilon_{py} - \varepsilon_y}}{\dfrac{Q_y}{\varepsilon_y}} \right)$;

Q_{py} is the load at yielding of prestressing steel ($= (f_{py} - \sigma_{pe}) A_p + f_{sy} A_s$);

f_{py} is the yield strength of prestressing steel;

A_p is the cross-sectional area of prestressing steel;

A_s is the cross-sectional area of steel reinforcement;

ε_{py} is the strain at yielding of prestressing steel $\left(= 0{,}002 + \dfrac{f_{py} - \sigma_{pe}}{E_p} \right)$;

σ_{pe} is the effective stress of prestressing steel;

E_p is the elastic modulus of prestressing steel;

Q_y is the virtual yield load of member $\left[= Q_{cr} + \dfrac{Q_{py} - Q_{cr}}{\varepsilon_{py} - \varepsilon_{cr}} \left(\varepsilon_y - \varepsilon_{cr} \right) \right]$;

Q_{cr} is the cracking load of member $\left[= f_t \left(A_c + \frac{E_s}{E_c} A_s + \frac{E_p}{E_c} A_p \right) \right]$;

A_c is the cross-sectional area of concrete;

Q_e is the elastic response axial tensile force;

ε_{ra} is the circumferential strain limit determined from securing of watertightness.

The strain limit shall be determined to be restorable to a level ensuring watertightness after unloading of seismic forces. This is required to be appropriately established referring to the results of tests and past studies, but the yield strain of reinforcement may generally be adopted as the strain limit when no data are available.

When using precast concrete members having joints across which the reinforcement is not continuous, an allowable strain for enabling restoration to a level of ensuring watertightness shall be appropriately established referring to the results of tests and past studies.

Investigation for θ = 180° is sufficient for a PC water tank of a general shape.

— Investigation of vertical bending moment

$$\frac{M_{ud}}{M_d} \geq \gamma_i \quad (54)$$

where

M_{ud} is the design flexural capacity;

M_d is the vertical bending moment of member determined by elastic analysis under the seismic load calculated using the design horizontal seismic coefficient, K_{h2};

γ_i is the structure factor (= 1,0).

Investigation for θ = 180° is sufficient for a PC water tank of a general shape.

The design vertical cross-sectional flexural capacity shall be determined based on the following assumptions:

a) The design cross-sectional flexural capacity according to the action direction of the member force with regard to the member cross-section or unit width shall be calculated based on the following assumptions:

— Fibre strain is proportional to the distance from the neutral axis.

— The tensile stress of concrete is disregarded.

— The stress-strain curve of concrete is as a rule based on Figure 15.

— The stress-strain curve of steel is as a rule based on Figure 16.

b) The distribution of the compressive stress of concrete may be assumed to be rectangular (equivalent stress block) as shown in Figure 14, excepting the case where the strains within a member cross-section are totally compressive or tensile.

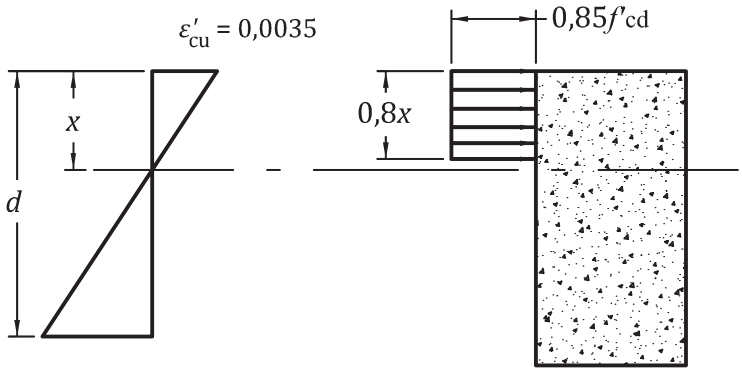


Figure 14 — Equivalent stress block

c) Stress-strain curve

1) Concrete

$$f'_{cd} = f'_{ck} / \gamma_m \tag{55}$$

where

f'_{cd} is the design compressive strength of concrete;

f'_{ck} is the characteristics compressive strength of concrete;

γ_m is the material factor.

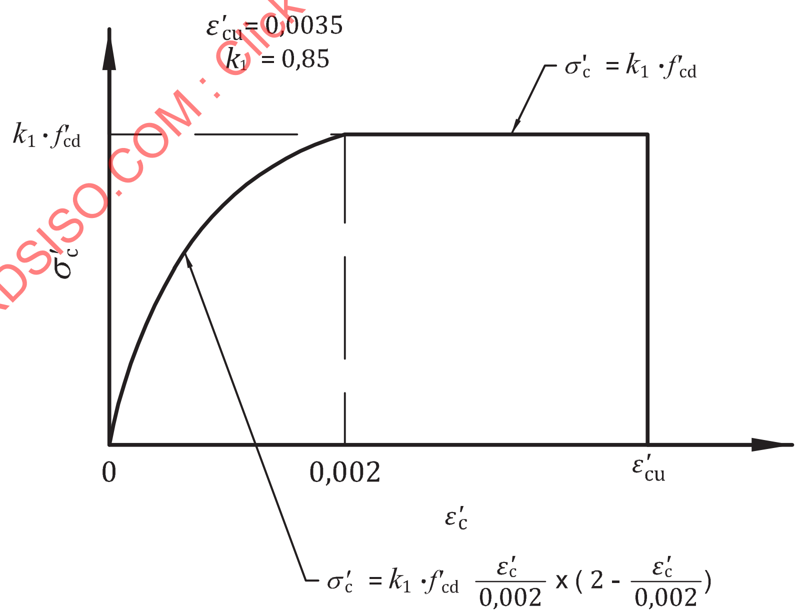


Figure 15 — Modelled stress-strain curve of concrete for calculating flexural capacity

2) Steel

$$f_{yd} = f_{sy} / \gamma_m \tag{56}$$

where

f_{yd} is the design yield strength of reinforcement and structural steel;

f_{sy} is the yield strength of reinforcement and structural steel;

γ_m is the material factor.

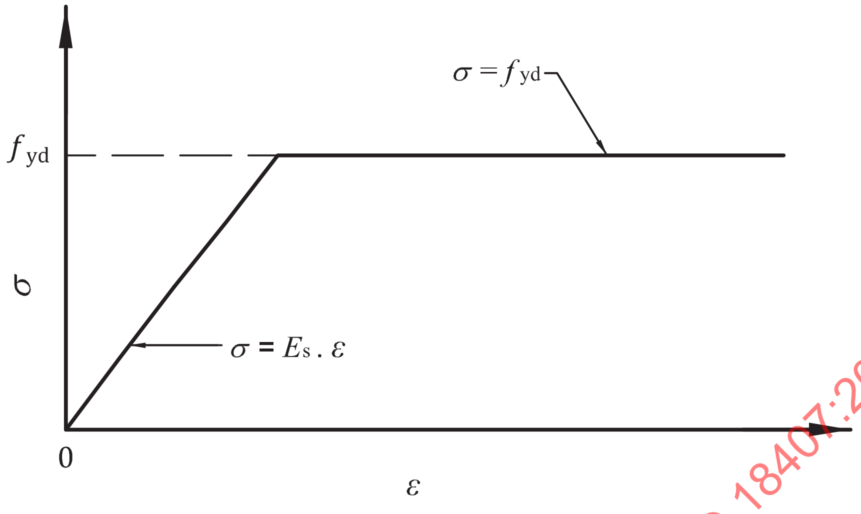
$$f_{ud} = f_{pu} / \gamma_m \tag{57}$$

where

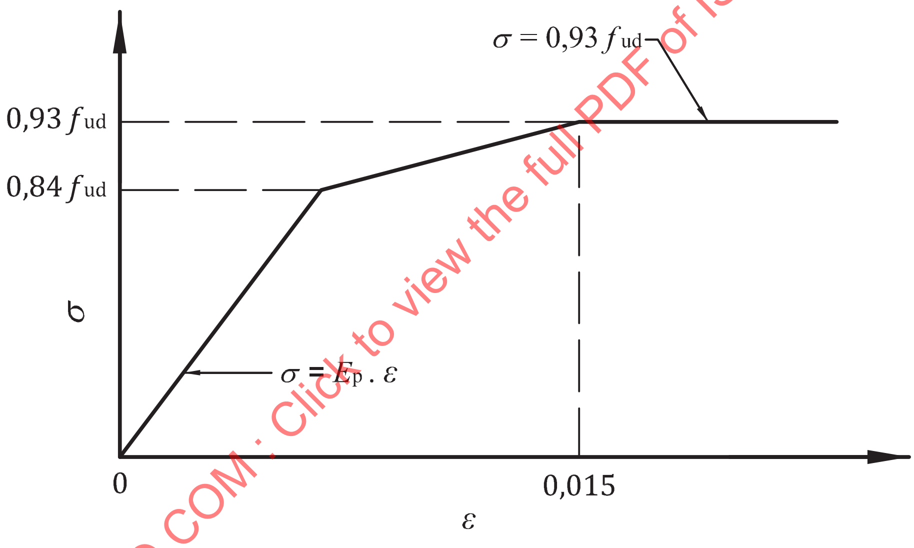
f_{ud} is the design tensile strength of prestressing steel;

f_{pu} is the tensile strength of prestressing steel;

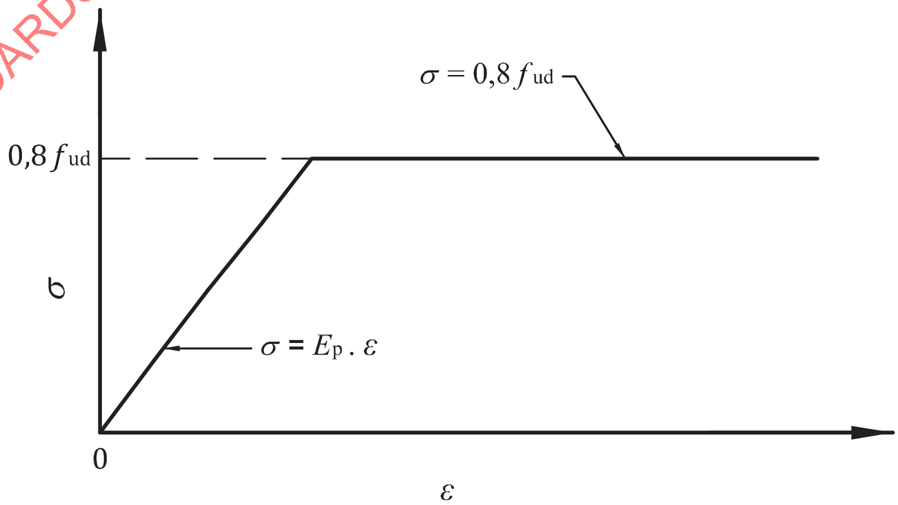
γ_m is the material factor.



a) Stress-strain curve of steel reinforcement and structural steel



b) Stress-strain curve of prestressing wires, prestressing strands and tri-linear stress-strain type prestressing bars



c) Stress-strain curve of bi-linear stress-strain type prestressing bars

Figure 16 — Modelled stress-strain curves of steel

d) Safety factor

1) The material factor, γ_m, may be the values given in Table 11.

Table 11 — Material factor

	γ_m
Concrete	[1,3]
Steel	[1,0]

2) The member factor, γ_b, may be set at [1,1].

When the prestressing steel is unbonded, it is advisable to reduce the design cross-sectional capacity of both members subjected to axial tensile forces and members subjected to bending moment and axial forces by 30 %.

In regard to the base slab of a PC water tank of a general shape, attainment of the earthquake resistance level specified in 9.1.3 shall be confirmed by confirming that the design flexural capacity is greater than the design bending moment. Specifically, this may be verified by the same method as Formula (54).

9.4 Investigation for foundation

Stability calculation of the foundation used for a PC water tank shall be carried out in consideration of the effect of an earthquake specified in 9.3.1.

In regard to the stability calculation of spread foundations, the design shall be done by the seismic coefficient method with respect to Level 1 ground motion, but verification is not necessary for the horizontal bearing capacity retained during an earthquake with Level 2 ground motion.

In pile foundation, design with respect to Level 1 ground motion shall be done by the seismic coefficient method. Design with respect to Level 2 ground motion shall be done by the method considered with plastic deformability of pile foundation. In this case, the seismic coefficient can be also adopted to decide the suitable seismic coefficient with Level 2 ground motion. For verification of pile foundation respect to Level 2 ground motion, the response of plastic deformation of pile foundation shall be reduced in consideration with the plastic deformability. The limitation of plastic deformability ratio of pile foundation shall be approximately set at [4]. For simplicity, the seismic safety of Level 1 ground motion can be verified by replacing the standard horizontal seismic coefficient of Level 1 ground motion with the standard horizontal seismic coefficient of Level 2 ground motion multiplying to 0,5.

10 General structural details

10.1 Prestressing steel

10.1.1 Clear distance

The clear distance between sheaths for post-tensioning shall be as specified as follows:

— the horizontal and vertical clearance between sheaths shall be not less than 4/3 times the maximum size of coarse aggregate;

— the horizontal clearance between sheaths or sheath groups where an internal vibrator is to be inserted shall be not less than 60 mm and shall be such that a space necessary for inserting an internal vibrator is secured;

— where it is inevitable, up to two lines of small circumferential sheaths may be placed in contact with each other. The number of such sheath groups shall be limited to two or less;

— it is advisable that the minimum clearance between circumferential sheaths or sheath groups is the vertical size of each sheath or sheath group.

In the case of wrapped prestressing, the clearance between prestressing steel shall be not less than 10 mm.

In the case of pretensioning, the clearance between prestressing steel at the ends of members shall be not less than three times the diameter of prestressing steel in both horizontal and vertical directions, and the clearance in the horizontal direction shall be not less than 4/3 times the maximum aggregate size. Also, when prestressing steel is placed in contact with each other in areas other than the ends of members, the total number of such prestressing steel groups shall be limited to four or less in not more than two circumferential rows. The clearance between such prestressing steel groups shall be not less than 4/3 times the maximum aggregate size.

In the case where the space between prestressing steel is significantly wide, the problem of stress concentration due to prestressing steel shall be considered.

When analysing stress due to circumferential prestressing steel, the loads due to prestressing steel are generally dealt with as distributed loads, but actually these are concentrated loads by prestressing steel. For this reason, it is necessary to consider the effect of stress concentration due to prestressing steel when the space between prestressing steel becomes significantly wide due to the use of particularly large diameter of prestressing steel.

Prestress propagates with a spread angle of approximately 45°. Care should therefore be exercised, as the specified prestress may not be applied when the distance from the anchorage to the design cross-section is small and the prestressing steel are widely spaced.

The maximum space of prestressing steel should preferably be not more than the following values:

Circumferential $l_{max} \leq 5t$

Vertical $l_{max} \leq 3t$

where

l_{max} is the maximum prestressing steel spacing;

t is the tank wall thickness.

10.1.2 Concrete cover

In the case of post-tensioning, concrete cover for sheaths shall meet the requirements given in Table 12.

Table 12 — Concrete cover

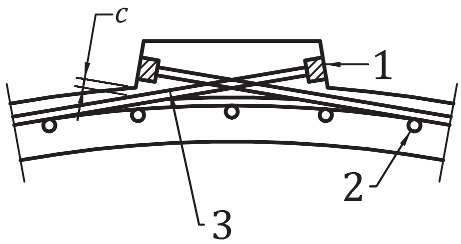
Sheath arrangement	Cover requirement
Distributed	≥[40] mm and not smaller than the horizontal size of each sheath
Two small sheaths (upper and lower) in contact with each other	≥[40] mm and not less than the horizontal size of each sheath group

In the case of wrapped prestressing, prestressing steel shall be covered with a covering material to a thickness of not less than [40] mm.

Special corrosion-inhibiting treatment shall be applied to the ends of pre-tensioning steel.

In the case of factory products, the values given in Table 12 may be reduced by up to 20 %.

The details of anchorages of circumferential prestressing steel are generally as shown in Figure 17. The size of pilasters shall be established so that the cover depth, c, for the sheath at the point shown in the figure would be sufficient.



Key

1 anchoring device

2 vertical sheath

3 circumferential sheath

Figure 17 — Cover concrete for sheaths near a pilaster

10.1.3 Arrangement of curved prestressing steel

When prestressing steel is curved, except in special case, the radius shall be so determined that the loss in prestress is as small as possible and that the bearing stress acting on the concrete is within acceptable limits.

10.1.4 Arrangement of anchorages and couplers

— Anchorages shall be arranged so that the prestress required for each design cross-section is effectively applied. Also, couplers shall be arranged so that prestressing steel is securely connected.

— When a number of anchorages are arranged in the same cross-section, the cross-sectional shape and size of concrete in the anchorage zone shall be established in consideration of the number and force of anchorages and minimum intervals required between adjacent anchorages.

10.1.5 Protection of anchorage zone

Anchorage of prestressing steel shall be protected to ensure that it is free from damage and corrosion during the design life of the structure.

It is preferable to anchor circumferential prestressing steel outside of the tank wall. However, when they are anchored on the inside for aesthetic or other reasons, particular care shall be exercised to protect the anchorage zones from water ingress. Care shall also be taken when anchoring vertical prestressing steel in the middle of the tank wall, as an anchorage could be a bleeding channel.

10.1.6 Reinforcement of concrete near anchorages

— Concrete near anchorages shall be reinforced with U-shaped or spiral steel reinforcement against the tensile stress generated perpendicularly to prestressing steel;

— When placing an anchorage in the middle of a member, concrete near the anchorage shall be reinforced with steel reinforcement against tensile stress generated in the concrete.

10.2 Steel reinforcement

10.2.1 Clear distance

— Clearance between main reinforcement or between main reinforcement and a sheath shall be as specified in Table 13.

— Clearance between horizontal or vertical steel reinforcement arranged in the form of a grid in the tank wall shall be less than 0,3 m.

Table 13 — Clearance between main reinforcement or between main reinforcement and sheath

Rebar arrangement	Clearance
Rebars and sheaths are dispersed	Not less than 20 mm and not less than 4/3 times the maximum aggregate size and not less than the diameter of rebar
Two or more rebars or sheaths are placed together	Not less than 20 mm and not less than 4/3 times the maximum aggregate size and not less than the diameter of the rebar or the sheath

10.2.2 Concrete cover

Concrete cover shall conform to Table 14. Depending on the environmental conditions, these values shall be increased.

Table 14 — Concrete cover for steel reinforcement

Condition	Cover depth
Portions not in contact with water or soil and portions in contact with soil but protected by an effective protective layer	≥[30] mm
Portions in contact with water or soil	≥[40] mm

The cover depth for steel reinforcement near the bottom surface of the base slab should be not less than [40] mm where levelling concrete is present. Where there is no levelling concrete, the cover depth should be not less than [60] mm. However, when a foundation slab is provided under the base slab, the cover depth may be reduced to not less than [30] mm.

When a foundation slab is provided, cover concrete of base slab on the side of the foundation slab may be regarded as a portion not in contact with water or soil.

For a PC water tank located near the sea and requiring measures against chloride attack, the depth of cover concrete on the side affected by external chlorides should be increased by at least [20] mm from the values given in Table 14.

10.2.3 Bend configurations of reinforcement

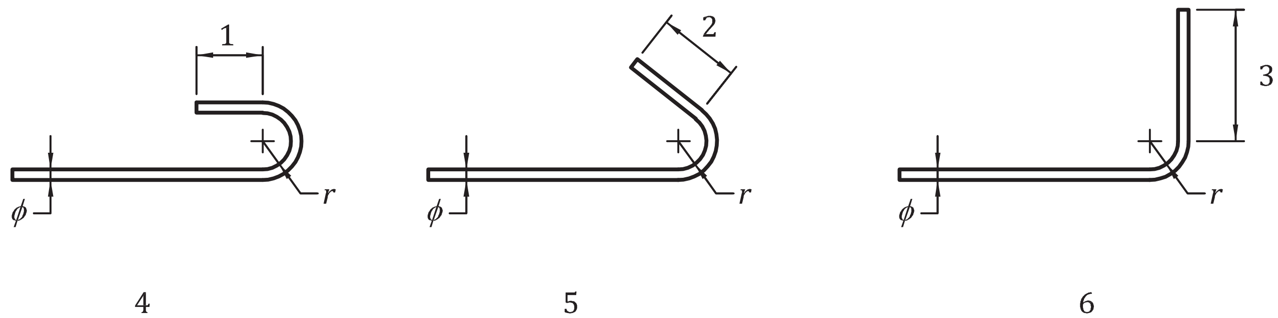
a) Standard hooks

Semicircular, rectangular, or acute-angled hooks shall be used as standard hooks.

— A semicircular hook shall have the end of the bar bent to 180° to form a semicircle and a straight extension that measures neither less than 4 times the bar diameter nor less than 60 mm (see Figure 18).

— A rectangular hook shall have the end of the bar bent to 90° and a straight extension that measures not less than 12 times the bar diameter (see Figure 18).

— An acute-angle hook shall have the end of the bar bent to 135° and a straight extension that measures not less than 6 times the bar diameter (see Figure 18).



Key

1 not less than 4ϕ nor 60 mm
2 not less than 6ϕ nor 60 mm
3 not less than 12ϕ
4 semicircular hook (for plain bar and ribbed bar)
5 acute-angle hook (for ribbed bar)
6 rectangular hook (for ribbed bar)
ϕ is the diameter of steel reinforcement
r is the inside radius bent

Figure 18 — Hook configuration at free ends of steel reinforcement

b) Longitudinal reinforcement

When using plain bars as longitudinal reinforcement, a semicircular hook shall be chosen as the standard hook at all times.

Bend radii for longitudinal reinforcement shall be not less than the values given in Table 15, where ϕ denotes the diameter of reinforcement.

Table 15 — Bend radii of hooks

Type (Yielding strength of steel reinforcement, MPa)		Bend radius of rebar hook (r) Longitudinal bar	Transverse bar
Plain bar	(235)	2,0 ϕ	1,0 ϕ
	(295)	2,5 ϕ	2,0 ϕ
Ribbed bar	(295)	2,5 ϕ	2,0 ϕ
	(345)	3,0 ϕ	2,5 ϕ
	(390)	3,5 ϕ	3,0 ϕ
	(490)	3,5 ϕ	3,0 ϕ

c) Transverse reinforcement (stirrups, ties and hoops)

— A standard hook shall be provided at the end of each transverse steel reinforcement. When plain bars are used for transverse reinforcement, the hooks shall be semicircular hooks.

— When ribbed bars are used for transverse reinforcement, semicircular or acute-angle hooks shall as a rule be provided at their ends.

— Bend radii for the hooks of transverse reinforcement shall be not less than the values given in Table 15.

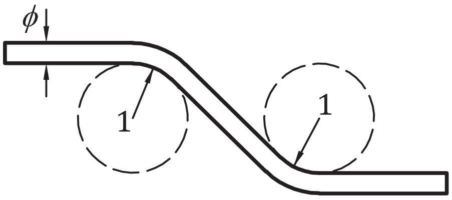
d) Other reinforcement

— Bend radii for bent-up bars shall be not less than 5 times the bar diameter (see Figure 19).

— When a bar within a distance of (20ϕ + 20) mm from the surface of a concrete member is used as a bent-up bar, the bend radius shall be not less than 7,5 times the bar diameter.

— Inside radii for reinforcement along the outer side of a corner in a rigid frame shall not be less than 10 times the bar diameter (see Figure 20).

— Reinforcement along the inner side of a haunch or a corner in a rigid frame shall not be tensile reinforcement bent from the slab or beam, but shall be separate straight reinforcement placed along the inner side of the haunch (see Figure 21).

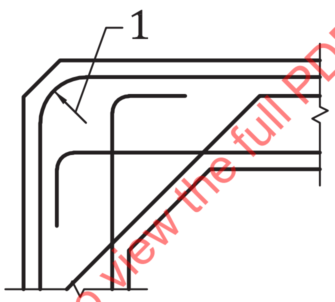


Key

1 not less than 5ϕ

NOTE ϕ is the diameter of reinforcement

Figure 19 — Bend radius of bent-up bar



Key

1 not less than 10ϕ

NOTE ϕ is the diameter of reinforcement

Figure 20 — Bend radius for reinforcement along the outer side of a corner in a rigid frame

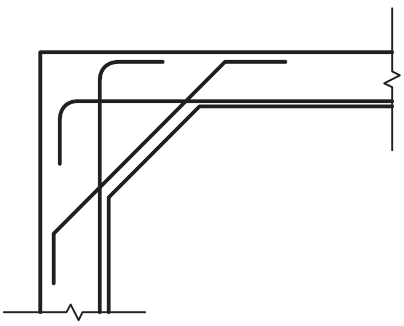


Figure 21 — Reinforcement along the inner side of a haunch corner in a rigid frame

10.2.4 Splices in reinforcement

a) General

— Appropriate splices shall be selected for steel reinforcement taking account of such factors as the type and diameter of the reinforcement, state of stress and locations of splices.

— Splices shall be arranged avoiding cross-sections subjected to high stress to the extent that it is possible.

— Multiple splices in the same cross-section shall be avoided. To prevent different splices from entering the same cross-section, the standard longitudinal distance between staggered splices shall be not less than the splice length plus 25 times the bar diameter.

— The clearance between two adjacent splices or between a splice and reinforcement shall be not less than the maximum aggregate size.

— When steel reinforcement is to be spliced after arrangement, a sufficient clearance shall be secured for the operation of jointing machines.

— Concrete cover for splices shall meet the requirements of 10.2.2.

b) Lap splices

Lap splices for longitudinal steel reinforcement shall meet the following requirements.

— In the case where (a) the reinforcement is two times or more of the ratio required by design calculations and (b) the ratio of spliced bars to the total reinforcement in a cross-section is 1/2 or less, the lap shall be not less than the basic development length, l_d.

— When either of conditions (a) and (b) above is not satisfied, the lap shall be not less than 1,3 l_d and lap splices shall be reinforced with transverse bars or other means.

— When neither (a) nor (b) is satisfied, the lap shall be not less than 1,7 l_d and lap splices shall be reinforced with transverse bars or other means.

— For lap splices to be subjected to low-cycle fatigue, the lap shall be not less than 1,7 l_d with hooks and lap splices shall be reinforced with spiral bars, metal fitting for reinforced connection, or other means.

— The lap of reinforcement splices in an underwater concrete structure shall as a rule be not less than 40 times the diameter of reinforcement.

— The length of lap splices shall be not less than 20 times the diameter of reinforcement.

10.2.5 Anchoring of reinforcement

The end of steel reinforcement shall be sufficiently embedded in the concrete so as to achieve the required stress of the reinforcement by bond between the bar and concrete, the provision of a hook, or a mechanical anchorage.

— Semicircular hooks shall be provided at the ends of plain bars at all times.

— At least 1/3 of the positive moment reinforcement in slabs or beams shall be extended beyond the support point without bending.

— At least 1/3 of the negative moment reinforcement in slabs or beams shall be extended beyond the folding point and be either embedded in the compression zone or continued to the neighbouring negative moment reinforcement.

— As to a bent bar, it is advisable to use its extension as positive or negative moment reinforcement, or to embed the bar in the compression zone with a minimum cover depth by extending its end toward the upper or lower surface of the beam and bending the bar horizontally and parallel to the surface.

— The ends of transverse reinforcement shall be provided with semicircular or acute-angle hooks enclosing longitudinal reinforcement.

— The ends of spiral bars shall be embedded in the concrete confined in the spiral bar with one and half extra turns.

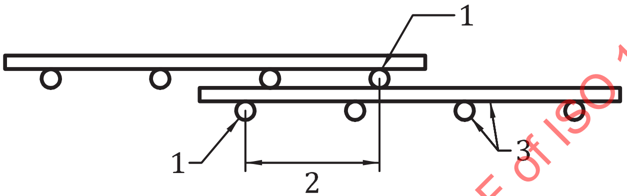
— When anchorage is to be achieved by bond between concrete and reinforcement or by a hook, the end of the bar shall be embedded with the specified development length beyond the critical section.

10.2.6 Welded wire fabric

The lap of spliced welded wire fabrics, which is defined as the distance between their outermost transverse reinforcement, shall be the interval between transverse bars plus 50 mm or more, but not less than 150 mm (see Figure 22).

When splicing welded wire fabrics having different transverse bar intervals or splicing welded wire fabric with normal reinforcing bars, the longer lap requirement shall be adopted.

The development length for welded wire fabric at fixed ends of members, which is defined as the distance between the surface of the supporting member and the outermost transverse bar, shall be the interval between transverse bars plus 50 mm or more, but not less than 150 mm (see Figure 23).



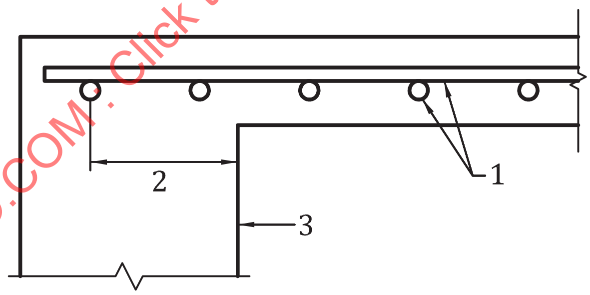
Key

1 outermost reinforcement

2 length of lap splice

3 welded wire fabric

Figure 22 — Lap splice of welded wire fabric



Key

1 welded wire fabric

2 development length

3 surface of supporting member

Figure 23 — Development length for welded wire fabric

10.3 Concrete joints

10.3.1 Construction joints

The locations and directions of construction joints shall be established in consideration of the strength and watertightness of the structure, as well as concrete placing efficiency. Appropriate measures are necessary to increase the watertightness at construction joints, as these joints are particularly prone to cause water leakage.

Measures to increase the watertightness of construction joints may include the following:

— after placing concrete, carefully wire-brush the joint surface to remove foreign particles before the concrete is fully set;

— insert a waterstop made of metal, rubber, etc., in the joint;

— apply an adhesive on the joint surface;

— waterproof the inside of the tank across the construction joints.

10.3.2 Joints between precast concrete members

When unifying separately fabricated precast members by using prestress, the locations and structure of joints shall be thoroughly examined so that the structure or the members can develop the required strength. The watertightness of the joints shall also be examined.

The angle between a joint surface and the resultant compressive force acting on the joint shall be 90°. When this angle is between 70° and 55°, the joint surface shall be appropriately treated. When this angle is between 55° and 45°, appropriate interlocking shall be provided on the joint surface. This angle shall not be 45° or less.

Joints to be connected using an adhesive shall be configured so that the joint surfaces can attach tightly to each other.

A prestressing steel to be placed in a joint between precast concrete members having no tensile reinforcement shall be bonded and part of the prestressing steel shall be located near the tension edge.

The width of a concrete joint should generally be greater than the length of a bar splice and steel reinforcement should be spliced in the joint. On the other hand, the width of a mortar joint should generally be as small as practicable and steel reinforcement shall not be spliced in the joint.

10.4 Reinforcement for opening

Openings in the roof, wall, or base slab shall be reinforced to ensure safety against cracking due to stress concentration or other causes. Particular care is necessary for openings in regions in contact with water so as not to cause water leakage.

It is advisable to avoid openings in the tank wall, which is subjected to large stress due to water pressure and prestress, to the extent that it is practicable. Instead, openings should be located in the base slab.

When an opening in the tank wall is inevitable, the opening shall be reinforced based on particularly careful investigation into the state of stress concentration.

An opening should be located in a region with relatively small stress and should generally be reinforced as shown in Figure 24.

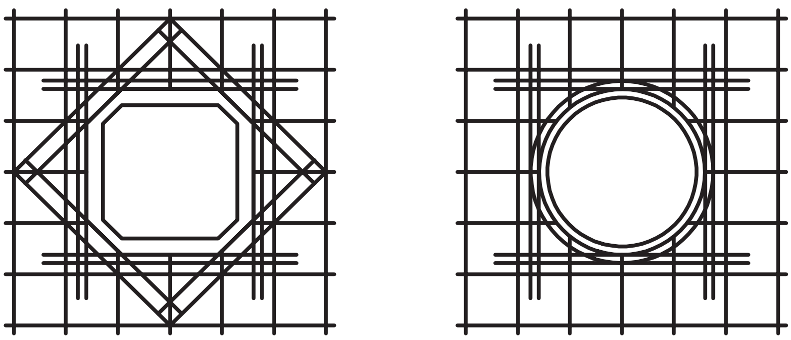


Figure 24 — Typical examples of reinforcement for openings

11 Design of members

11.1 Method of calculating member force

11.1.1 Analysis method

The analysis method for calculating the member force shall be selected from those that allow analysis of thin-shell structure, with thorough consideration to the structural characteristics of the PC water tank.

Specifically, analysis methods include the following:

— Analysis method by the finite element method (FEM) using axisymmetric thin-wall shell elements: this method may be used, as computer-aided FEM analysis has been generally used for structural analysis of the tank;

— Method based on elastic shell theoretical solutions: where the components of the tank, such as the roof, wall and base slab, are deemed analysable as single bodies like a PC water tank of a general shape, analysis may be carried out by design methods using conventional formulas based on elastic shell theoretical solutions;

— Other methods: other methods include an analysis method in which the axisymmetric structure is modelled into a plane frame problem.

11.1.2 Analysis model

The model to be used for analysis to calculate the member forces may be the following:

— A model that deals with such components as the roof, wall and base slab as part of a total system

This model is schematically shown in Figure 25. Analysis using such a model is generally based on FEM using axisymmetric thin-wall shell elements. Other choices may include a model in which the roof is separately analysed as a single body while the tank wall and base slab are unified, as well as a model in which the base slab is separately analysed as a single body while the roof and tank wall are unified.

The effects of the ground below the base slab are generally evaluated by modelling into distributed vertical and horizontal springs. In such a case, each spring constant should be established by an appropriate method.

Axisymmetric loads may be evaluated by assuming vertical springs to be the vertical subgrade reaction modulus determined from the characteristics of the ground, K_V. However, the application of generally used K_V, which is premised on the use for a relatively small loading width and rigid footing, as it is, to the base slab of a PC water tank, which is premised on deformation with a relatively large loading width, can lead to underestimation of K_V. When the tank wall is fixed-supported at the bottom, if such an underestimated K_V is used as vertical springs for analysis in a model having a unified tank wall and base slab, then the bending moment at the wall bottom can be extremely underestimated. Actual measurement of tanks has also revealed that the moment may not be as small as the analysis value. Sufficient care should therefore be exercised for the evaluation of K_V. The values given in Table 16 may be used as the vertical subgrade reaction modulus, K_V, for a PC water tank of a general shape.

Horizontal springs for the analysis may be evaluated as the horizontal shear subgrade reaction modulus determined from the characteristics of the ground, K_S. For a PC water tank of a general shape, K_S may be evaluated as 1/3 to 1/4 of K_V.

For a structure with a tank wall fixed-supported by the base slab, when the wall and base slab are modelled as unified while the effect of the ground below the base slab is modelled into springs, care should be exercised in the following case: If the vertical bending moment at the wall bottom is significantly lower than the value obtained from a model with completely fixed wall-bottom connection by such a factor given in Table 21 or by 0,7, it is advisable to examine by testing whether or not the modelling of the effect of the ground is appropriate. Note that the factors given in Table 21 correspond to the case of using the values given in Table 16.

Table 16 — Vertical subgrade reaction modulus, K_V

		Normal ground	Firm bedrock
K_V (N/cm^3) when the support condition at the tank wall bottom is “fixed”	Ordinary	100	1 000
	During an earthquake	200	2 000

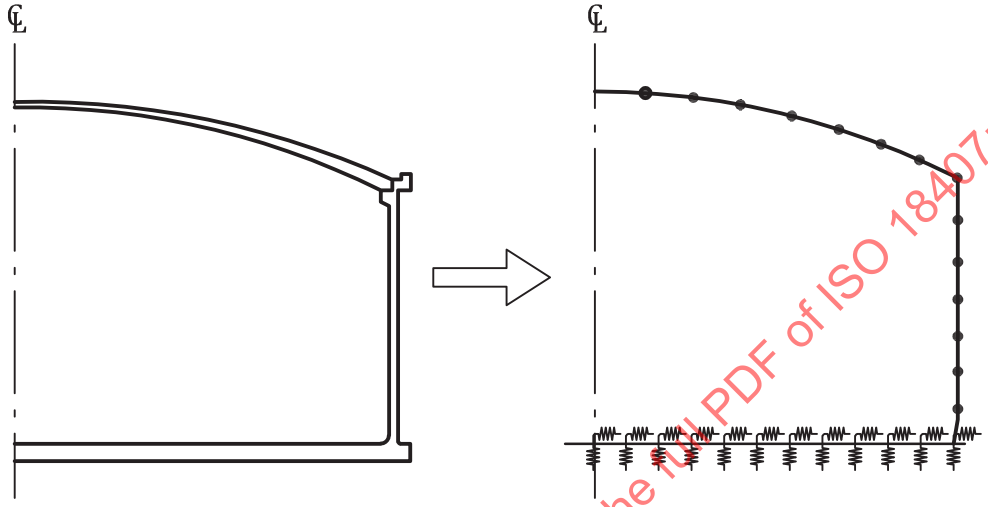


Figure 25 — A model assuming the components to be part of the total system

— A model that deals with such components as the roof, wall and base slab as single bodies

This model is schematically shown in Figure 26. Analysis using such a model includes FEM with axisymmetric thin-wall shell elements and a method based on theoretical solutions. However, when assuming the tank wall fixed to the base slab to be “completely fixed” disregarding the effect of the base slab, the member force of the tank wall should be corrected in accordance with 11.4.2.6 including the case of FEM using axisymmetric shell elements.

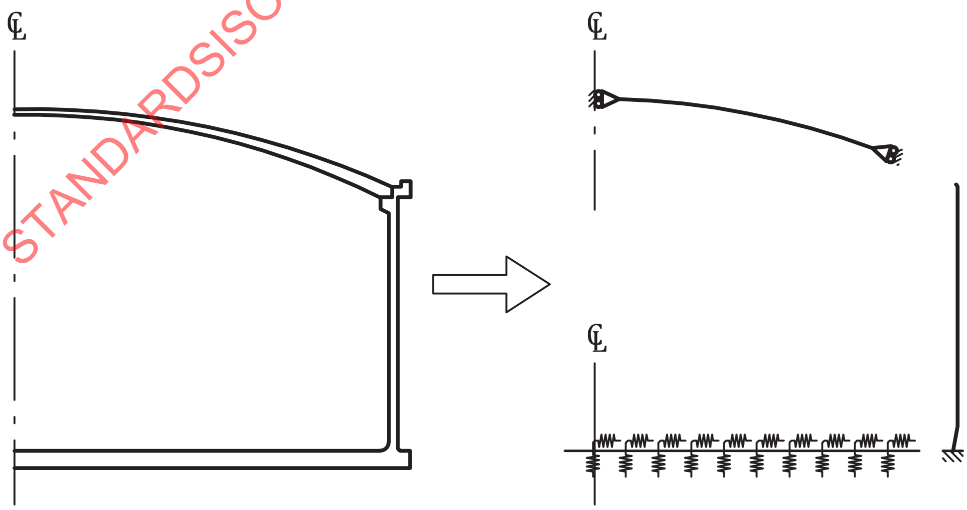


Figure 26 — Model assuming the components to be separate bodies

11.2 Component division

A PC water tank comprises three members: the roof, wall and base slab. The definition of each member shall be as follows:

Roof: A cover built on top of the tank wall, including the dome ring of a spherical dome.

Tank wall: A cylindrical shell built on the base slab. Excepting fixed support, the bearing is included in the tank wall.

Base slab: A slab constructed at the bottom of the tank wall, including the upper surface of the ground.

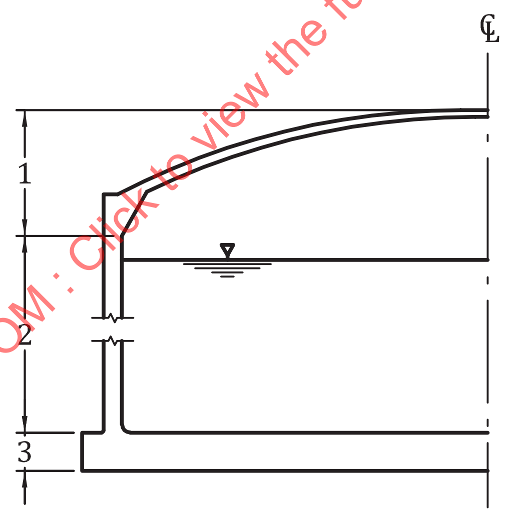
Simple division may pose problems, as the members of a PC water tank are somehow related to one another. However, the members are divided as shown in Figure 27 according to their general names.

The performance requirements for each member are as follows:

Roof: The roof is a member that shall be constructed to prevent external water, dust, trash, etc. from entering the tank containing drinking water, as well as to protect the water from sunlight. The water contained in the tank should not come into contact with the roof while in use.

Tank wall: The tank wall is a structural member that shall be constructed to store water. Watertightness and strength are required.

Base slab: A member that shall resist the deformation and subsidence of the foundations under loads from above, while retaining watertightness.



Key

1 roof

2 wall

3 base slab

Figure 27 — Component division

11.3 Roof

11.3.1 Structural types

The structural types of the roof include the following:

— spherical dome (spherical shell) structure;

— slab structure;

— other structure.

NOTE 1 A spherical dome is the most general type of construction for the roof of a PC tank with the largest number of examples. The bottom of the dome is generally supported by the tank wall without building columns. The connection between the tank wall and roof is mostly fixed or hinged as schematically shown in Figure 28.

NOTE 2 A slab includes flat slab construction supported by intermediate columns and a method configuring a slab on a framing made of beams. It is a structural form effective for the cases where there are height limitations and where the tank is to be filled back.

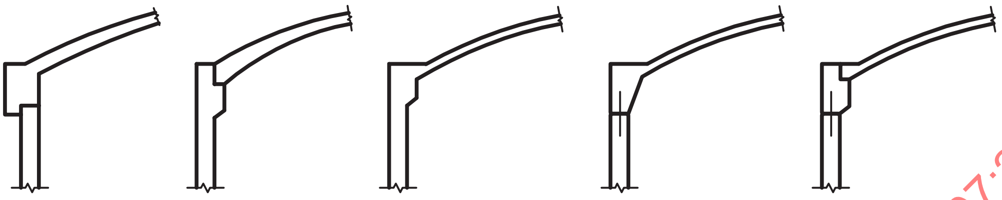


Figure 28 — Spherical dome construction

11.3.2 Design in general

11.3.2.1 Loads for roof design

The loads to be used for roof design shall be as follows:

— deadweight in accordance with 6.2;

— imposed load in accordance with 6.3;

— prestressing force;

— snow load where applicable in accordance with 6.10;

— other loads.

These loads shall be considered for roofs of any form in normal design. Other loads such as the effects of wind and an earthquake, as well as the effects of drying shrinkage and temperature are not necessary to consider excepting certain special cases.

It is not necessary to consider the prestressing force to be applied to the tank wall, as it scarcely affects the dome.

11.3.2.2 Design of a spherical dome

The design shall be based on elastic spherical shell analysis or other generally accepted techniques. The analysis of a spherical dome includes membrane analysis that deals with only the membrane stress of the dome shell and bending analysis that considers the support conditions of the dome bottom.

In a PC water tank of a normal dome roof type, prestressing forces corresponding to the horizontal thrust forces are applied to the dome ring after placing concrete for the dome. For this reason, the stress generated in the dome is equal to the membrane stress. Investigation of only membrane stress is therefore required.

The following method may be used as the basic formulae for calculating the membrane force, in which definition of the symbols is indicated in Figure 29:

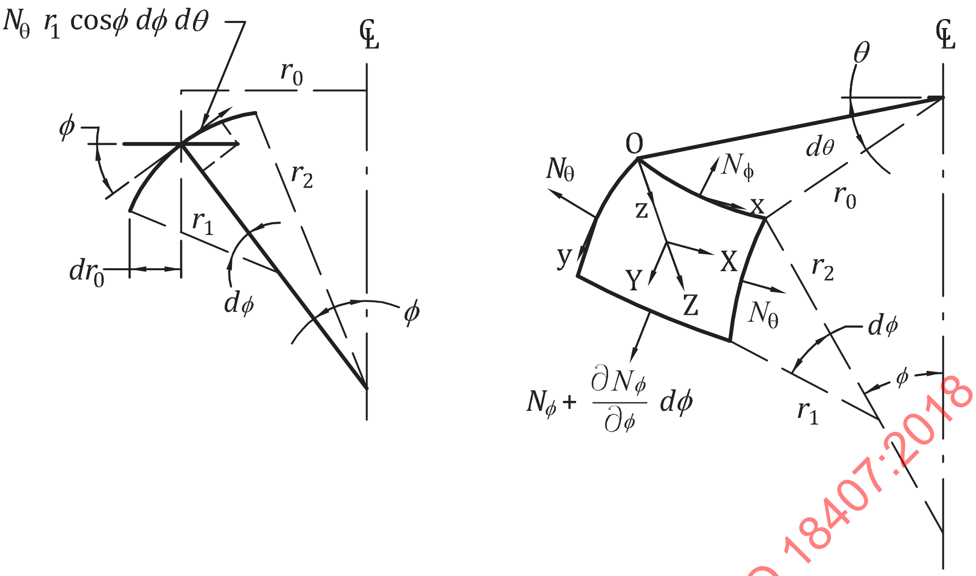


Figure 29 — Elements and forces of an axisymmetric shell

Equilibrium formulae

$$\frac{d}{d_\phi}\left(N_\phi r_0\right) - N_\theta r_1 \cos\phi + Y r_1 r_0 = 0$$
$$N_\phi r_0 + N_\theta r_1 \sin\phi + Z r_1 r_0 = 0 \tag{58}$$

In the case of a spherical dome, N_θ and N_ϕ are obtained by solving Formula (58) while assuming $r_1 = r_2 = r$. Figures 30 and 31 schematically show membrane forces by deadweight and by imposed load and snow load, respectively.

$$N_{\phi d} = \frac{r q_d}{1 + \cos\phi_d}$$
$$N_{\theta d} = r q_d \left(\cos\phi_d - \frac{1}{1 + \cos\phi_d} \right) \tag{59}$$

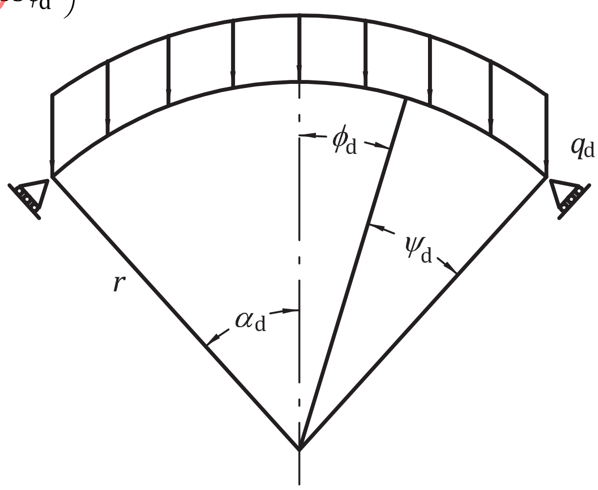


Figure 30 — Membrane force by deadweight

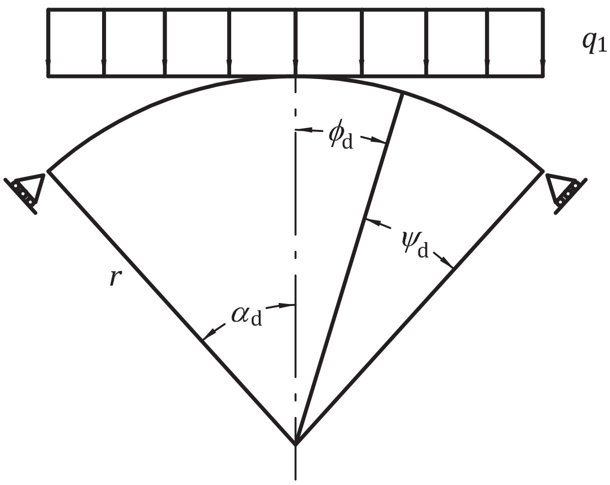


Figure 31 — Membrane force by imposed load and snow load

$$N_{\phi d} = \frac{rq_l}{2}$$
$$N_{\phi d} = \frac{rq_l}{2} \cos 2\phi_d \qquad (60)$$

where

$N_{\phi d}$ is the membrane force per unit length of the dome in the meridian direction;

$N_{\theta d}$ is the membrane force per unit length of the dome in the parallel direction;

r is the dome radius;

α_d is a half angle of the dome;

ϕ_d is the angle from the rotation axis at an arbitrary point of the dome;

ψ_d $= \alpha_d - \phi_d$;

q_d is the deadweight per unit area;

q_l is the imposed load per unit area.

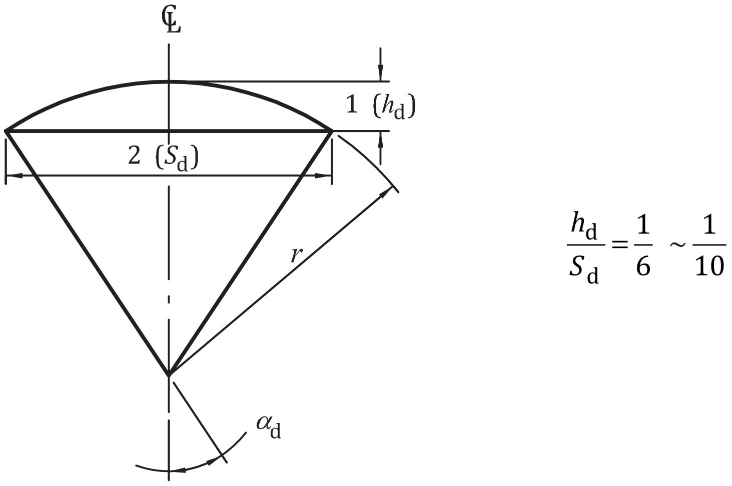
In normal calculation, Formulae (59) and (60) lead to similar results. Therefore, the treatment of all loads using Formula (59) is allowed so that the prestressing force for the dome ring would be sufficient. Also, Formula (60) deals with uniform distribution of snow.

Bending analysis shall be carried out when prestress for the dome ring is to be applied before concrete placing and when the prestressing force widely differs from the horizontal thrust due to snow loads. Also, when considering complicated loads, such as large eccentric snow loads under special geographical conditions, it is advisable to use a generally accepted technique, such as FEM.

— The minimum thickness of the dome shall be 0,1 m in consideration of the post-buckling stability of a spherical dome and placeability of concrete.

— The rise-span ratio shall be 1/6 to 1/10 as a standard as shown in Figure 32.

NOTE A larger rise-span ratio reduces the horizontal thrust but makes the construction work more difficult. On the contrary, a smaller rise-span ratio makes the construction work easier but increases the horizontal thrust, increasing the prestressing force on the dome ring.



Key

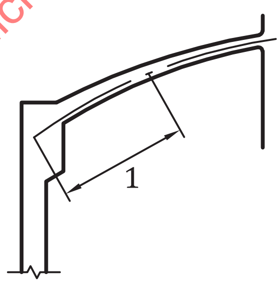
1 rise (h_d)

2 span (S_d)

Figure 32 — Rise-span relationship

— The minimum reinforcement content of the dome in which ribbed bars are to be used shall be 0,25 % of the cross-sectional area both in the parallel and meridian directions. When using plain bars, a value 0,05 % greater than the above-mentioned value should be adopted.

— The marginal area of the dome that is within 5° from the dome edge shall be thickened and reinforced, as shown in Figure 33, with double reinforcement, unless bending analysis is conducted to investigate the member cross-section.



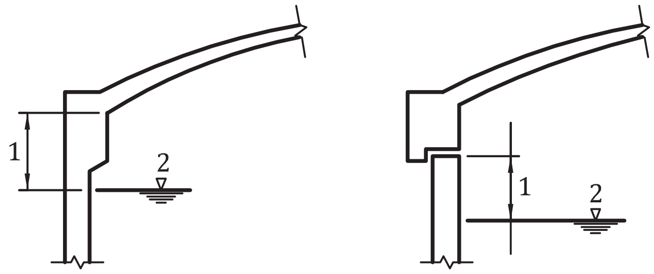
Key

1 thickened section (not less than 5°)

Figure 33 — Dome edge to be thickened

— The clearance height shall be not less than 0,3 m (See Figure 34).

NOTE The clearance height is determined so that the dome would not be subjected to uplift pressure even when the water level tentatively exceeded the estimated high-water level due to malfunction of the valve system or incorrect operation. The standard clearance height is given in Table 17 for reference.



Key

1 clearance height is not less than 0,3 m

2 H.W.L.

Figure 34 — Clearance height

Table 17 — Clearance height depending on capacity

	Capacity (m^3)				
	1 000	5 000	10 000	20 000	30 000
Clearance (m)	0,5	0,6	0,7	0,8	0,85

11.3.2.3 Design of the dome ring

— A dome ring shall be provided around the dome margin for spherical dome (spherical shell) construction.

— For a spherical dome, a prestressing force corresponding to the horizontal thrust force acting on the dome margin should be applied to the dome ring.

A calculation of horizontal thrust due to the deadweight of the dome and prestressing force is given in Figure 35.

Surface area of the dome:

$$A_d = 2\pi r h_d = 2\pi r^2 \left(1 - \cos\alpha_d\right) \tag{61}$$

Deadweight of dome:

$$W_d = A_d q_d \tag{62}$$

Vertical load per unit length in the circumferential direction at dome margin:

$$V_d = \frac{W_d}{\pi S_d} \tag{63}$$

Horizontal thrust:

$$H_t = \frac{V_d}{\tan\alpha_d} \text{ or } H_t = N_{\phi d} \cos\alpha_d \tag{64}$$

Prestressing force to be applied to dome ring:

$$F_d = H_t \frac{S_d}{2} \tag{65}$$

It is advisable that the prestressing steel be placed at symmetrical positions above and below Point G shown in Figure 36.

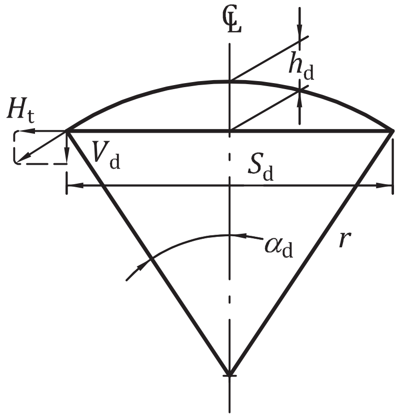
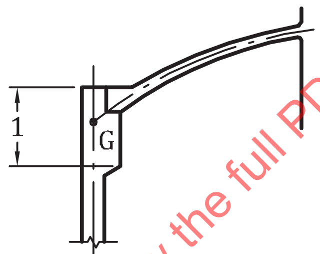


Figure 35 — Horizontal thrust



Key

1 dome ring

G intersection of the dome centre and wall centre

Figure 36 — Dome axis line

It is advisable that the prestressing steel be placed at symmetrical positions above and below Point G shown in Figure 36.

— The minimum reinforcement in the circumferential direction of the dome ring shall be not less than 0,25 % of the cross-sectional area.

NOTE The minimum reinforcement specified is intended to address the effects of shrinkage, temperature, etc. before applying the prestress.

— When a large snow load is considered, the axial stress and other stress of the dome ring shall be investigated. If the total horizontal thrust is counterbalanced by prestress, then a large compressive force remains in the dome ring in summer. Also, if the total horizontal stress is not counterbalanced by prestress, then an axial tensile force is generated. Care should therefore be exercised about the large imbalance between the horizontal thrust due to the load on the dome and the horizontal component of the prestressing force.

11.3.2.4 Design of slab roof

When designing a slab roof, the member force shall be calculated in consideration of the structural form. Slab roofs are mostly flat slabs generally having columns and combinations of beams and slabs. Analysis methods for these roofs are mostly based on flat slabs or rigid frames depending on the structural form. The thickness of these slabs shall be not less than 0,15 m and it is advisable to determine the details referring to other recommendations. When columns are used, investigation is necessary for the joints between the roof and the columns regarding bending moment and shear forces. The clearance shall be not less than 0,3 m (see Figure 34).

11.4 Tank wall

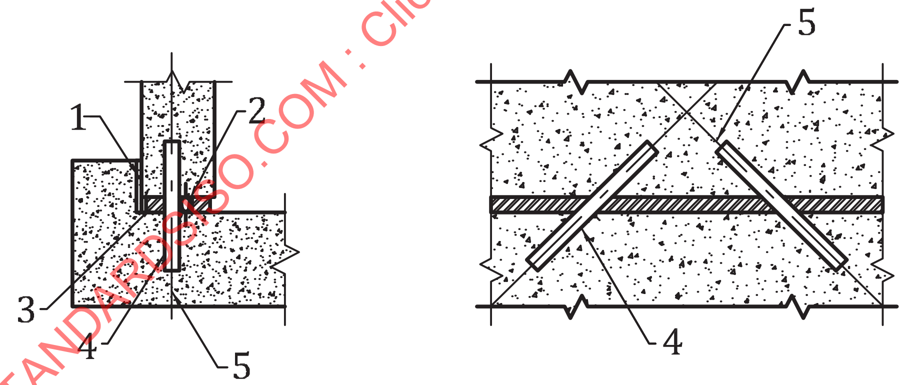
11.4.1 Structural types

The support types of tank wall bottoms generally include freely sliding support, hinged support and fixed support. The watertightness and earthquake resistance are highest with fixed support, followed by hinged support and freely sliding support in this order. However, the order is reversed from the aspect of not restricting the characteristics of shell stress.

It is difficult to construct a supporting structure at the bottom of a tank wall in an ideal freely sliding, hinged, or fixed condition. Sufficient care should therefore be exercised from the aspects of both design and execution so that the structural characteristics of each support form would be retained.

Freely sliding support is a type of joint between the bottom of a tank wall and the base slab that allows rotation and horizontal displacement of the tank wall with respect to the base slab. An ideal freely sliding support would not cause bending moment in the vertical direction, but actually the deformation restraint of the bearing material causes bending moment. Appropriate measures should therefore be taken. This is also referred to as “elastic joint” due to the characteristics of the bearing. Figure 37 shows a typical example of this structure. For a freely sliding support, a tank wall is generally constructed on a rubber shoe placed on the base slab as shown in Figure 37 to allow radial deformation using the shear deformation of the rubber shoe. In this case, special earth quake-proof anchors (earth quake-proof cables) are used to bear the shear forces during an earthquake.

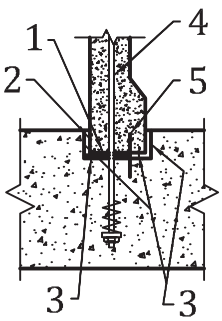
Hinged support is a type of joint between the bottom of a tank wall and the base slab that allows only rotation of the tank wall with respect to the base slab. The tank wall is generally constructed on a rubber shoe placed on the base slab and anchors are used to restrain radial and circumferential displacements. Steel reinforcement or prestressing steel is used as the anchors. Figure 38 shows typical examples of hinged support.



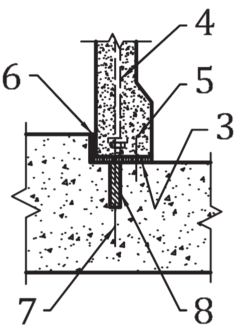
Key

1 caulking
2 waterstop
3 rubber shoe
4 sleeve
5 earthquake-proof anchors

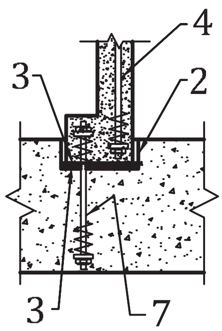
Figure 37 — A typical example of freely sliding support



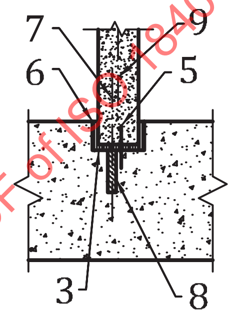
a) Prestressing bar fixed in the base slab (the bottom wall is not always thickened)



b) Anchor used with rebar (vertical structure of the tank wall is prestressed concrete)



c) Anchor used prestressed bar



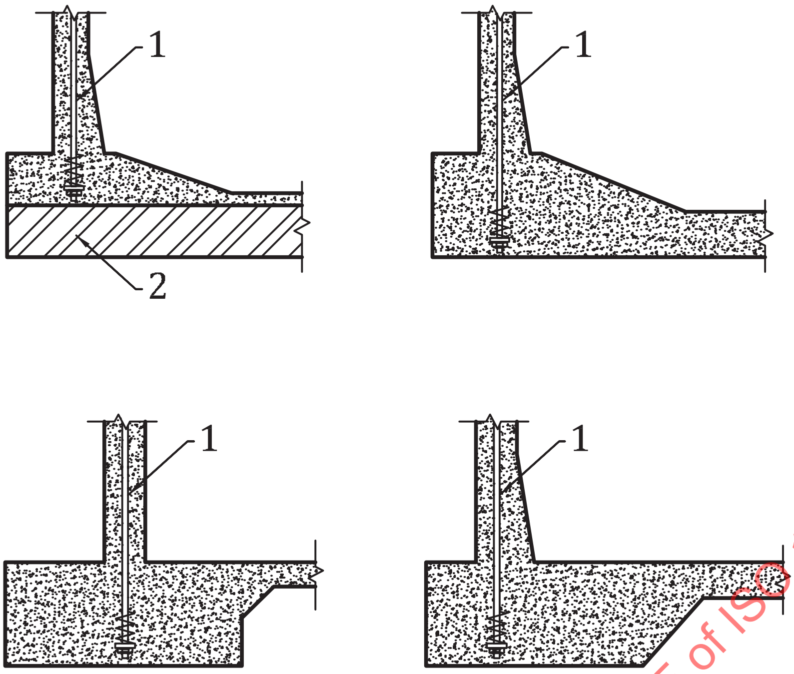
d) Anchor used with rebar (vertical structure of the tank wall is reinforced concrete)

Key

1 rubber shoe
2 caulking
3 rubber
4 prestressing bar
5 waterstop
6 elastic filler
7 anchor (rebar or prestressing bar)
8 asphalt
9 rebar

Figure 38 — Typical examples of hinged support

Fixed support is a type of joint between the tank wall and the base slab that allows no rotation or horizontal displacement of the tank wall with respect to the base slab. Since a small rotation is actually generated in the base slab, this may also be referred to as “elastic fixing.” Figure 39 shows typical examples of fixed support.



Key

1 prestressing bar

2 foundation slab

Figure 39 — Typical examples of fixed support

11.4.2 Design in general

11.4.2.1 Loads for tank wall

Loads to be used for the design of a tank wall shall include:

— deadweight, this refers to the deadweight of the roof and tank wall;

— imposed load in accordance with 6.3;

— hydrostatic pressure in accordance with 6.4;

— prestressing force;

— effects of creep and drying shrinkage;

— effect of temperature;

— effect of an earthquake, with 6.8 and Clause 9;

— wind load in accordance with 6.9;

— snow load in accordance with 6.10;

— earth pressure, with 6.11;

— other loads.

Prestressing force refers to the prestressing force corresponding to the hydrostatic pressure + residual compression strength in the circumferential direction and the prestressing force to cope with the bending moment in the vertical direction.

If the structural system remains unchanged, then it is not necessary to consider the effect of creep. The bottom of the wall and base slab of a normal PC water tank are constructed with no appreciable gap between the construction periods and subjected to similar environmental conditions after completion. Since the effect of drying shrinkage is similar on both members, investigation is unnecessary except in the case under special conditions.

The effects of temperature may include the effect of uniform temperature rise/drop and the effect of temperature difference between the inside and outside of the tank wall. Investigation into the effect of uniform temperature rise/drop is unnecessary. Temperature differences between the inside and outside do not last long but occur more frequently than uniform temperature changes.

The temperature stress due to temperature difference between the inside and outside is generally calculated by Formula (66).

$$\sigma_c = \pm \frac{T \alpha_e E_c}{2(1-\upsilon)} \tag{66}$$

where

σ_c is the stress of concrete (the surface with a lower temperature causes tensile stress; "+" represents compressive and "−" represents tensile);

T is the temperature difference (refer to 6.7);

α_e is the linear expansion coefficient of concrete (may generally be set at 10×10^{-6}/°C);

E_c is the elastic modulus of concrete;

ν is the Poisson's ratio.

Where a large temperature difference is expected between the inside and outside of the tank wall, it is advisable to use earth fill or insulation to reduce the difference.

For a tank partially covered with earth fill, the earth pressure shall not be considered in a "tank full" condition. This is because earth pressure due to earth fill acts in the direction opposite to the water pressure, resulting in a compressive force in the circumferential direction. Though the effect of normal earth fill is small, it should be investigated in a "tank empty" condition.

Other loads may include those during construction. Certain construction methods may tentatively involve excessive stress.

11.4.2.2 Combinations of loads

Combinations of loads for the design of a tank wall shall generally be as given in Table 18.

Table 18 — Load combinations

			Immediately after prestressing	Under ordinary conditions		Under temp. action	During an earthquake
				Empty	Full	Full	
Primary load	Deadweight		○	○	○	○	○
	Imposed load			○	○	○	○
	Hydrostatic pressure				○	○	○
	Prestressing force	Immediately after	○				
		Effective		○	○	○	○
	Effect of creep			○	○	○	○
Subsidiary load	Effect of an earthquake						○
Particular load	Effect of temperature					○	
	Snow load			○	○	○	○
	Earth pressure			○			

The effect of creep and particular loads shall be considered as necessity arises.

Loads other than those given above shall also be considered if they have a strong effect on the tank wall.

11.4.2.3 Design of the tank wall

The basis of design shall be the elastic cylinder shell theory or other generally accepted method. The elastic cylinder shell theory is described with the symbols defined in Figure 40 as follows:

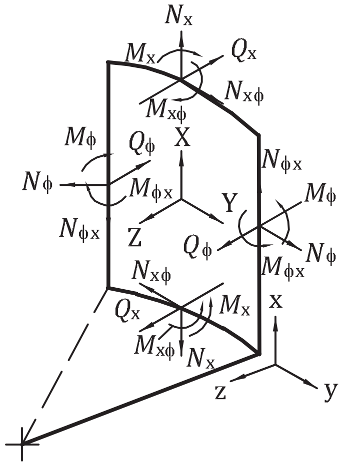


Figure 40 — Member forces in a cylindrical shell

Key

N_x	is the vertical axial force
N_ϕ	is the circumferential axial force
M_x	is the vertical bending moment
M_ϕ	is the circumferential bending moment
$N_{x\phi}$, $N_{\phi x}$	are the in-plane shear forces
$M_{x\phi}$, $M_{\phi x}$	are the torsional moments
Q_x, Q_ϕ	are the out-of-plane shear forces
X, Y, Z	are the external forces in x, y and z directions, respectively

When dealing with axisymmetric loads, $N_{x\phi} = N_{\phi x} = M_{x\phi} = M_{\phi x} = Q_\phi = 0$.

— Basic formula and solution

From the equilibrium of member forces shown in Figure 40, the basic formula is expressed as Formula (67):

$$\frac{d^4 w_x}{dx^4} + 4\beta^4 w_x = \frac{Z}{K} \tag{67}$$

where

w_x is the radial displacement determined by Formula (69);

x is the distance from wall bottom;

β is the characteristic value of the tank wall $\left(= \sqrt[4]{\frac{Et}{4R^2K}} \right)$;

E is the elastic modulus;

t is the tank wall thickness;

R is the radius;

K is the flexural rigidity $\left[= \dfrac{Et^3}{12\left(1-\nu^2\right)} \right]$;

ν is the Poisson's ratio.

Also,

$$M_x = -K \frac{d^2 w_x}{dx^2}$$

$$Q_x = -K \frac{d^3 w_x}{dx^3} \tag{68}$$

$$N_\phi = -\frac{Et}{R} w_x$$

The solution of Formula (67) is expressed by Formula (69):

$$w_x = C_1 \sin\beta x \sinh\beta x + C_2 \sin\beta x \cosh\beta x + C_3 \cos\beta x \sinh\beta x + C_4 \cos\beta x \cosh\beta x + f\left(x\right) \tag{69}$$

where

C_1 to C_4 are the integration constants determined from the boundary conditions of the cylinder end;

$f(x)$ is the particular solution of Formula (67).

— Displacements and member forces due to M_0 and Q_0

As shown in Figure 41, the displacements and member forces when restraining moment, M_0, and restraining shear force, Q_0, act on the bottom of the cylindrical shell (x = 0) are as given below.

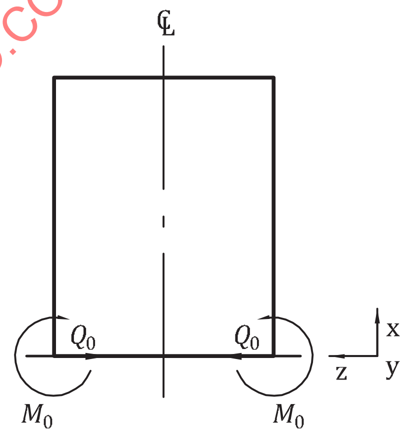


Figure 41 — M_0 and Q_0 acting on tank wall bottom

In this case, no external forces are distributed over the shell surface and the $f(x)$ in Formula (69) is zero. Therefore, the solution is Formula (70). The boundary condition when M_0 and Q_0 act is given in Formula (71).

$$w_x = C_1 \sin\beta x \sinh\beta x + C_2 \sin\beta x \cosh\beta x + C_3 \cos\beta x \sinh\beta x + C_4 \cos\beta x \cosh\beta x \tag{70}$$

$$\begin{aligned}
&-K\left(\frac{d^2 w_x}{dx^2}\right)_{x=0} = M_0 \\
&-K\left(\frac{d^3 w_x}{dx^3}\right)_{x=0} = Q_0 \\
&-K\left(\frac{d^2 w_x}{dx^2}\right)_{x=H} = 0 \\
&-K\left(\frac{d^3 w_x}{dx^3}\right)_{x=H} = 0
\end{aligned} \tag{71}$$

where

M_0 is the restraining moment at the bottom;

Q_0 is the restraining shear force at the bottom.

By substituting Formula (71) into Formula (70), the integration constants, C_1 to C_4, are determined as follows:

$$\begin{aligned}
&C_1 = -\frac{M_0}{2\beta^2 K} \\
&C_2 = \frac{\beta M_0 X_2(2\phi) + Q_0 Y_1(2\phi)}{2\beta^3 K} \\
&C_3 = \frac{\beta M_0 X_2(2\phi) + Q_0 Y_2(2\phi)}{2\beta^3 K} \\
&C_4 = \frac{\beta M_0 X_1(2\phi) + Q_0 X_3(2\phi)}{2\beta^3 K}
\end{aligned} \tag{72}$$

where

$$X_1(2\phi) = \frac{\cosh 2\phi - \cos 2\phi}{\cosh 2\phi + \cos 2\phi - 2}$$

$$X_2(2\phi) = \frac{\sinh 2\phi + \sin 2\phi}{\cosh 2\phi + \cos 2\phi - 2}$$

$$X_3(2\phi) = \frac{\sinh 2\phi - \sin 2\phi}{\cosh 2\phi + \cos 2\phi - 2}$$

$$Y_1(2\phi) = \frac{1 - \cos 2\phi}{\cosh 2\phi + \cos 2\phi - 2}$$

$$Y_2(2\phi) = \frac{\cosh 2\phi - 1}{\cosh 2\phi + \cos 2\phi - 2}$$

$$2\phi = 2\beta H$$

Therefore, by substituting C_1 to C_4 expressed as Formula (72) into Formula (70), the displacement under restraining moment, M_0, and restraining shear force, Q_0, is determined. The member force is determined by differentiating Formula (70).

— Displacement and member forces due to distributed loads

When only external forces act on a freely sliding cylindrical shell, the displacement and member forces are as follows:

The solutions including particular solutions of the basic formula [Formula (67)] for a tank wall subjected to uniform triangular and rectangular distributed loads are as given in Table 19 by solving methods including the undetermined coefficient method.

Table 19 — Solutions to basic formulae for the case where external forces act on an unrestrained cylindrical shell

a) Triangular distributed loads (Prestress corresponding to water pressure, etc.)	b) Rectangular distributed loads (Residual compression, etc.)
$Z=\rho(H-x)$ $w_x=\dfrac{\rho(H-x)}{4\beta^4K}$ $\dfrac{dw_x}{dx}=-\dfrac{\rho}{4\beta^4K}$ $M_x=-K\left(\dfrac{d^2w_x}{dx^2}\right)=0$ $Q_x=-K\left(\dfrac{d^3w_x}{dx^3}\right)=0$	$Z=g_0$ $w_x=\dfrac{g_0}{4\beta^4K}$ $\dfrac{dw_x}{dx}=0$ $M_x=-K\left(\dfrac{d^2w_x}{dx^2}\right)=0$ $Q_x=-K\left(\dfrac{d^3w_x}{dx^3}\right)=0$

NOTE ρ = unit weight of water, g_0 = uniform pressure, H = total water depth of tank

— Displacements and member forces due to triangular and rectangular loads under each support conditions

w_x, M_x, Q_x and N_ϕ in an arbitrary cross-section of tanks with different support conditions can be calculated by Formula (73).

$$w_x=\frac{\rho(H-x)}{4\beta^4K}+\frac{g_0}{4\beta^4K}+C_1F_4+C_2F_3+C_3F_2+C_4F_1$$

$$M_x=-2\beta^2K(C_1F_1+C_2F_2-C_3F_3-C_4F_4) \tag{73}$$

$$Q_x=-2\beta^3K\{C_1(F_2-F_3)+C_2(F_1-F_4)-C_3(F_1+F_4)-C_4(F_2+F_3)\}$$

$$N_\phi=-\frac{Et}{R}w_x$$

where

$$F_1 = \cos(\beta x)\cosh(\beta x)$$

$$F_2 = \cos(\beta x)\sinh(\beta x)$$

$$F_3 = \sin(\beta x)\cosh(\beta x)$$

$$F_4 = \sin(\beta x)\sinh(\beta x)$$

In these equations, M_0 and Q_0 to be substituted into Formula (72) to determine C_1 to C_4 in Formula (73) should be determined by the formulae given in Table 20 according to respective support conditions.

In these calculations, $M_0 = M_{01} + M_{02}$ and $Q_0 = Q_{01} + Q_{02}$ are assumed.

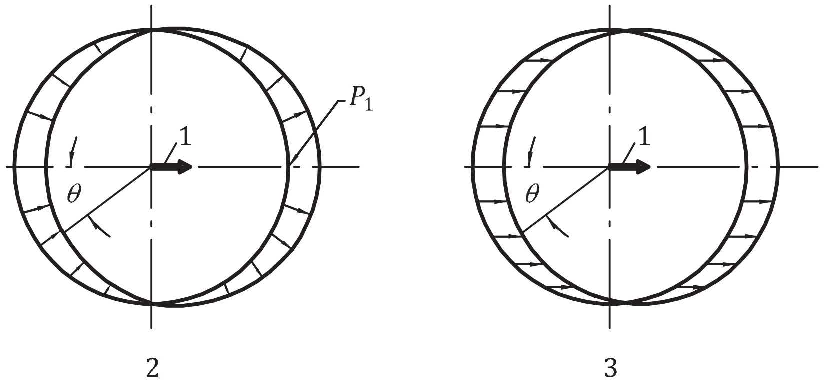
M_0 and Q_0 in Table 20 are obtained by applying respective support conditions in this table to Formula (73).

Table 20 — M_0 and Q_0 obtained from support conditions at the bottom of the tank wall

Load support condition	Triangular load (Prestress corresponding to water pressure, etc.)	Rectangular load (Residual compression, etc.)
Freely sliding	$M_{01} = 0$ $Q_{01} = 0$	$M_{02} = 0$ $Q_{02} = 0$
Hinged	$M_{01} = 0$ $Q_{01} = \dfrac{\rho H}{2\beta X_3(2\phi)}$	$M_{02} = 0$ $Q_{02} = \dfrac{g_0}{2\beta X_3(2\phi)}$
Fixed (Gradient of displacement curve)	$M_{01} = -\dfrac{\rho H}{2\beta^2}\dfrac{X_1(2\phi) - X_3(2\phi)/(\beta H)}{2X_2(2\phi)X_3(2\phi) - [X_1(2\phi)]^2}$ $Q_{01} = \dfrac{\rho H}{2\beta^2}\dfrac{2\beta X_2(2\phi) - X_1(2\phi)/H}{2X_2(2\phi)X_3(2\phi) - [X_1(2\phi)]^2}$	$M_{02} = -\dfrac{g_0}{2\beta^2}\dfrac{X_1(2\phi)}{2X_2(2\phi)X_3(2\phi) - [X_1(2\phi)]^2}$ $Q_{02} = \dfrac{g_0}{\beta}\dfrac{X_2(2\phi)}{2X_2(2\phi)X_3(2\phi) - [X_1(2\phi)]^2}$

When dealing with asymmetric loads, such as dynamic water pressure and inertia force, the equilibrium formulae of a shell are expressed by Formula (74) and Figure 42.

$$\begin{aligned}
& R\frac{dN_x}{dx} + \frac{dN_{\phi z}}{d\phi} + RX = 0 \\
& \frac{dN_\phi}{d\phi} + R\frac{dN_{x\phi}}{dx} - Q_\phi + RY = 0 \\
& R\frac{dQ_x}{dx} - \frac{dQ_\phi}{d\phi} + N_\phi + RZ = 0 \\
& R\frac{dM_{x\phi}}{dx} - \frac{dM_\phi}{d\phi} + RQ_\phi = 0 \\
& \frac{dM_{\phi x}}{d\phi} + R\frac{dM_x}{dx} - RQ_x = 0
\end{aligned} \tag{74}$$



$X = 0$

$Y = 0$

$Z = P_1 \cos\theta$

P_1: Dynamic water pressure at 180°

$X = 0$

$Y = K_\mathrm{h}\, \rho_\mathrm{c}\, t \sin\theta$

$Z = K_\mathrm{h}\, \rho_\mathrm{c}\, t \cos\theta$

K_h: design horizontal seismic coefficient

ρ_c: unit weight of wall

t: wall thickness

Key

1 acting direction of earthquake

2 dynamic water pressure

3 inertia force

Figure 42 — Load distributions during an earthquake

The minimum thickness of the tank wall shall be 0,2 m for embedding and 0,17 m for wrapping and precast concrete. Note that a minimum thickness of 0,2 m is adopted in view of the fact that reinforcement is basically of double arrangement and that a thickness of 0,25 m is normally desirable from the aspect of concrete placeability but ease of placing may be ensured by the use of self-compacting concrete, etc., in certain cases. The minimum thickness of a tank wall can be determined from the unit axial compressive stress limit of the tank wall due to circumferential prestress using Formula (75).

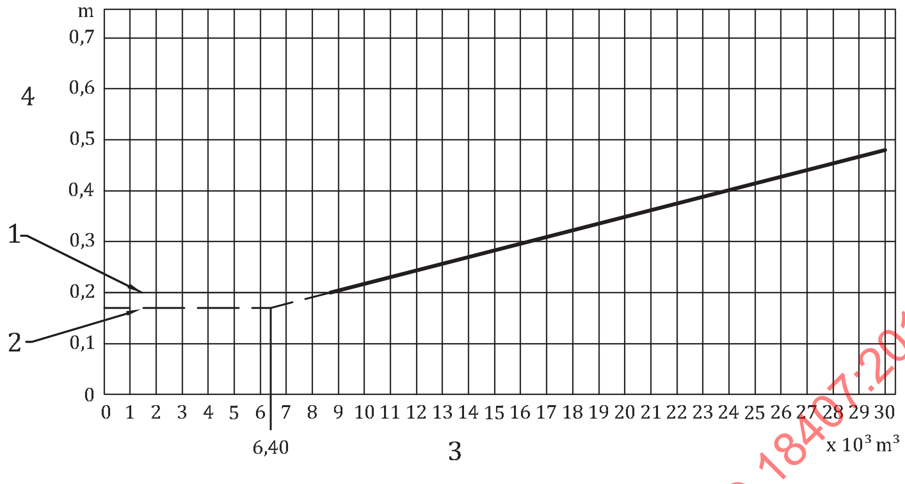
$$t_\mathrm{min} = \frac{N_\phi}{\sigma'_\mathrm{ca}} = \frac{\rho \gamma_0^{-\frac{1}{3}} \left(\frac{4V}{\pi} \right)^{\frac{2}{3}}}{2\sigma'_\mathrm{ca}} \tag{75}$$

where

- N_ϕ is the circumferential axial force;
- t_min is the minimum thickness of the tank wall;
- γ_0 is the internal diameter-water depth ratio (=D/H);
- ρ is the unit weight of water;
- σ'_ca is the axial compressive stress limit;
- V is the tank volume capacity.

Note that the residual compression was assumed to be 1,0 MPa.

Figure 43 shows the minimum wall thickness required for a cylindrical PC water tank with an internal diameter-water depth ratio of 3,0 to 3,5.



Key

1 embedded system

2 wrapped system and precast concrete

3 volume

4 wall thickness

Figure 43 — Minimum thickness required for tank wall

When using ribbed bars, the minimum reinforcement of the tank wall shall be 0,25 % of the cross-sectional area both in the circumferential and vertical directions. However, the minimum circumferential reinforcement content of ribbed bars in the zone from the bottom of the tank wall up to a height of 1,0 m or to the height of the haunch, whichever is higher, shall be 0,55 %. This ratio may be reduced to 0,35 %, if the tank wall is capable of following radial deformation. When plain bars are used, the ratio is required to be increased by 0,05 %.

If a tank with fixed support or hinged support is to be kept empty for a long time, the minimum reinforcement of ribbed bars of the PC tank shall be as follows:

— The minimum circumferential reinforcement content in the zone from the bottom of the tank wall up to a height of 1,0 m or to the height of the haunch, whichever is higher, shall be 1,5 % of the cross-sectional area.

— The minimum vertical reinforcement content shall be 0,25 % near the outer surface of the zone from the bottom of the tank wall up to a height of 1,0 m or to the height of the haunch, whichever is higher, in the case of fixed support and near the inner surface in the range of the point of maximum vertical bending moment ±1,0 m in the case of hinged support.

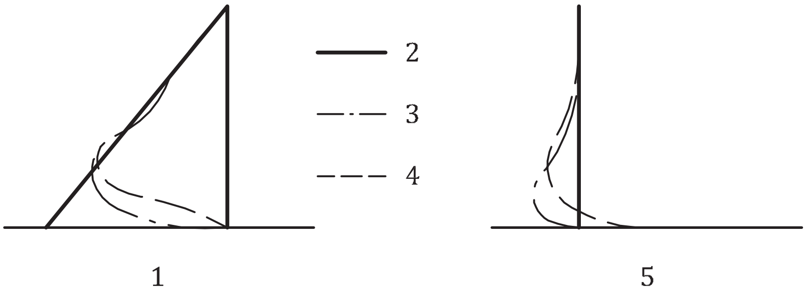
NOTE When the tank is to be kept empty for a long time, the progress of drying shrinkage in the tank wall may differ from that in the base slab, causing indeterminate forces at the bottom of the tank wall. This generates axial tensile forces in the circumferential direction in the bottom zone of the tank wall. Vertical moment, which increases the maximum moment in a "tank empty" condition, is also generated. Since it is difficult to properly determine this load in a quantitative manner, the requirement for the minimum reinforcement is adopted as a simple method of handling this.

When plain bars are to be used, the ratio should be increased by 0,05 %.

11.4.2.4 Method of applying prestress

Prestressing forces shall surely be applied to the tank wall in the circumferential direction.

Hydrostatic pressure causes hoop tension in the circumferential direction of the tank wall as shown in Figure 44.



Key

1 hoop tension

2 free

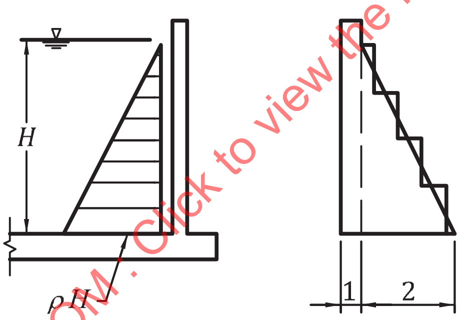
3 hinge

4 fix

5 vertical bending moment

Figure 44 — Hoop tension and vertical bending moment

Therefore, circumferential prestressing forces become necessary to counterbalance the generated hoop tension. The "sum of the force corresponding to the hydrostatic pressure and the excess compressive force (a force corresponding to 0,5 MPa to 1,0 MPa)" is generally applied as the prestressing force (see Figure 45).



Key

1 residual compression force

2 equivalent force to hydrostatic pressure

Figure 45 — A general method of applying prestress

However, the prestressing force may be determined based on detailed investigation according to the importance and scale of the structure, accuracy of structural analysis and the loads to consider.

Particular care shall be exercised where the structure at the bottom of the tank wall changes after prestressing.

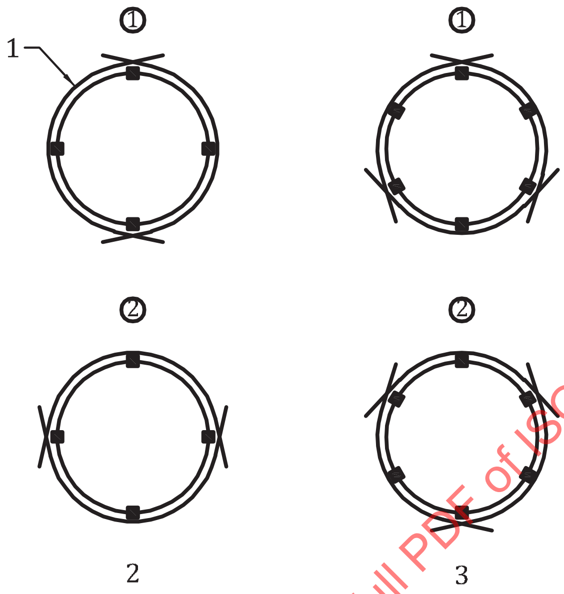
Methods of addressing vertical tensile stress include the application of prestress and the use of reinforcement as found in small-scale tanks and in the case where the bottom of the tank wall is freely sliding condition.

11.4.2.5 Pilasters

Pilasters for anchoring circumferential prestressing steel shall be equally spaced and their number shall be an even number not less than four.

The size of pilasters shall be established with due consideration to the cover depth and anchoring method.

In the case of embedding, the circumferential prestressing force does not become constant throughout the circumference due to various losses (Refer to 6.5). For this reason, it is advisable to arrange pilasters as shown in Figure 46, so as to equalize the prestressing force to the extent that it is possible. Four pilasters are mostly adopted for a PC water tank with a diameter of 20 m or less, whereas 6 or 8 are mostly adopted for 20 m or greater tanks.



Key

1 prestressing steel

2 case of 4 pilasters

3 case of 6 pilasters

NOTE 1 If anchored as ①, next prestressing steel is arranged as ②.

NOTE 2 Prestressing steel is arranged as the same way continuously at vertical direction.

NOTE 3 Mark ■ is a position of pilaster.

Figure 46 — Arrangement of pilasters and staggering of prestressing steel anchorages

When determining the pilaster size, it is advisable to refer to the guidelines and instructions for the specific anchoring method before making decision, as the configuration of anchorages varies depending on the anchoring system.

11.4.2.6 Design of the tank wall bottom

The bottom of the tank wall of a PC water tank shall be designed with due consideration to the support type.

The designer shall exercise sufficient care about the design of the bottom of a tank wall, as the stress conditions of the tank wall vary depending on the support type (see 11.4.1).

Ideal unrestrained support would not cause restraint at the bottom of the tank wall of a PC water tank, but actually restraining forces are generated due to shear resistance, friction resistance, etc., of the bearing material. An elastic bearing material (rubber padding) should be used at the bottom of the tank wall, so as to follow the deformation of the wall bottom in the radial direction due to creep and drying shrinkage, as well as elastic deformation. Also, seismic cables, such as prestressing strands, should be used to connect the tank wall and the base slab, so as to resist the base shear during an earthquake.

The shear resistance of seismic cables and the elastic bearing material (rubber padding) cause restraining forces that act on the bottom of the tank wall, generating small vertical bending moment in the tank wall. Care shall therefore be exercised so that this bending moment can be satisfactorily addressed. This vertical bending moment may be calculated based on Formula (76).

$$M_x = \frac{Q_0}{\beta}\zeta(\beta x) = \frac{kp}{k+2\beta^3 K}\frac{1}{2\beta^2}\zeta(\beta x) \quad (76)$$

where

$$Q_0 = \frac{pR^2}{Et}\frac{2\beta^3 Kk}{k+2\beta^3 K} = \frac{kp}{k+2\beta^3 K}\frac{1}{2\beta};$$

k is the spring constant of bearing;

p is the pressure at tank wall bottom (prestress, water pressure and earth pressure).

Sufficient care shall also be exercised, as this bending moment is strongly affected by the mechanical properties of the bearing material (e.g. hardness, shear modulus and compressive strain ratio of rubber padding).

Ideal hinged support would not cause bending moment at the bearing. However, bending moment could be generated by inadequate selection or use of the bearing material, prestressing steel material and shear connectors or inappropriate method of embedding them into the base slab. Sufficient care shall therefore be exercised so that the bearing performance requirements would be satisfied.

Prestressed concrete construction is mostly adopted for a hinge-supported tank to resist the vertical tensile stress, but reinforced concrete construction may be adopted for a tank involving relatively small bending moment (see 11.4.2.4).

Safety against base shear is secured by the shear resistance of prestressing steel or steel reinforcement to be used as anchorages or by the shear resistance of the base slab when the tank wall is to be placed into the base slab. However, the shear strength of prestressing steel tends to conversely decrease as the tensile stress increases. For this reason, shear force shall not be borne by tensioned prestressing steel, unless sufficient investigation is conducted regarding stress.

Sufficient care shall be exercised about corrosion inhibition of anchors, as anchors cannot be easily replaced.

Regarding fixed support:

— Effects of haunch at the bottom of the tank wall

In the case where the bending moment at the bottom of a tank wall with a fixed support condition is greater than that at mid-height, an increased thickness at the wall bottom is a choice to deal with this. In this case, the increase in the wall thickness shall be not more than the wall thickness. When a thickness increment greater than the wall thickness is necessary, it is preferable to increase the wall thickness itself.

An increase in the thickness of the wall bottom increases the rigidity of the wall, affecting its member forces. This effect is particularly strong on the vertical bending moment. When the increment in the wall thickness equals the original wall thickness, for instance, the vertical bending moment can nearly double the value determined based on uniform thickness. The effect of increase in the rigidity shall therefore be considered when increasing the thickness at the wall bottom. This effect can be evaluated by such methods as theoretical solutions, FEM and plane frame analysis. Otherwise, the following approximation may be used to calculate the vertical bending moment.

$$M_{0h} = \alpha_0 \frac{t_h}{t} M_{0c} \tag{77}$$

where

M_{0h} is the vertical bending moment at wall bottom considering an increase in the thickness at wall bottom;

M_{0c} is the vertical bending moment at wall bottom assuming a constant wall thickness, t;

t_h is the thickness of wall bottom;

t is the thickness of the tank wall;

α_0 is the correction factor $\left[= \frac{t}{t_h} + a_1 \beta H_h + a_2 \left(\beta H_h\right)^2 \right]$;

β is the characteristic value of the tank wall;

H_h is the length of thickened zone of the tank wall (haunch height).

Table 21 — Correction factors α_0 and α_p

	t_h/t	a_1	a_2
α_0	2,0	0,680	−0,161
	1,75	0,660	−0,188
	1,5	0,600	- 0,224
α_p	2,0	0,383	0,0807
	1,75	0,393	0,0308
	1,5	0,374	−0,0257
NOTE Haunch limitations: $1{,}5\ t \le t_h \le 2{,}0\ t$ $3{,}0\ (t_h - t) \le H_h \le 4{,}0\ (t_h - t)$			

A t_h/t value between the values given in Table 21 within the range of the table may be determined by interpolation.

Also, the values out of the range of this table may be calculated using Formula (77) after separately confirming the applicability of this formula by FEM, etc.

— Effects of elastic fixing

In regard to a fixed-supported PC water tank, when the rigidity of the base slab at the support is significantly greater than that of the tank wall, the joint shows a behaviour similar to that of completely fixed support. When there is no large difference between the rigidities of the base slab and tank wall, the joint shows a behaviour of elastic fixed support. The bending moment generated at the bottom of the tank wall in the latter case is smaller than that in the former case.

For a fixed-supported PC water tank that is expected to show an elastic behaviour, the bending moment at the bottom of the tank wall may be determined by Formula (78) unless a strict design method is adopted with consideration to the characteristics of foundations and the rigidity of the base slab.

$$M_{0f} = (k_\alpha k_\beta) M_{0h} \tag{78}$$

where

k_α is a coefficient considering the characteristics of foundations;

k_β is the coefficient considering the rigidity of base slab;

M_{0f} is the vertical bending moment at bottom of the tank wall.

Table 22 — Products of k_α and k_β

		$t_b = t_h$	t_b = 1,5 t_h	t_b = 2,0 t_h
Where the base slab can be regarded as one layer and of a uniform thickness	Normal ground	0,75	0,90	0,95
	Firm ground	0,80	0,90	0,95
Where the base slab can be regarded as two layers or the like	With a foundation slab	0,70	0,80	—
	Firm ground	0,70	0,80	—

The maximum thickness of the base slab (t_b) should not be greater than 2,0 times the thickness of the bottom of the tank wall (t_h) and that intermediate values within the range of Table 22 should be calculated by interpolation. Also, the formula is inapplicable to the case where a base slab that cannot be regarded as of a uniform thickness is placed on normal ground and the case of pile foundations.

— Effects of eccentricity of vertical prestressing steel

Eccentric arrangement of prestressing steel according to the distribution of bending moment is the basis of prestress design for bending moment. However, prestressing steel is generally placed in linear arrangement, due to the thin section of the wall of a PC water tank. When the vertical prestressing steel is eccentrically arranged throughout the tank wall height, sufficient care should be exercised, as the effect of eccentricity (eccentric moment) is limited to the area liberated from rotation.

An increase in the thickness of the tank wall at its bottom shifts the centroid of the cross-section away from the prestressing steel, generating a bending moment. This is a moment caused by the changes in the curvature radius due to the bend of the axis line and is determined depending on the increment in the tank wall thickness (haunch thickness) and length of the thickened wall. Techniques to accurately evaluate this effect include the theoretic solutions, FEM and plane frame analysis. When these techniques are not to be used, the following approximation may be used to determine the vertical bending moment:

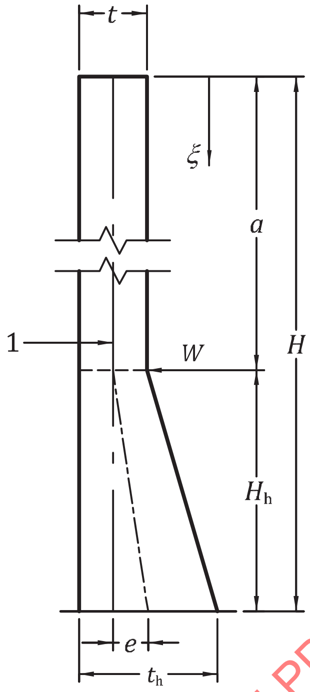
$$M_e = \alpha_p \frac{t_h}{t} M_{0e} \tag{79}$$

where

M_e is the vertical bending moment at tank wall bottom generated by changes in the curvature radius of the tank wall;

α_p is the correction factor (see Table 21) $\left[= \frac{t}{t_h} + a_1 \beta H_h + a_2 (\beta H_h)^2 \right]$;

M_{0e} is the vertical bending moment at tank wall bottom generated by changes in the curvature while the tank wall thickness is constant at t.



Key

1 prestressing bar

Figure 47 — Definition of symbols

M_{0e} is obtained by Formulae (80) and (81) with the symbols defined in Figure 47:

$$y = C_1 m_4(\beta\xi) + \frac{C_2}{2\beta}\{m_2(\beta\xi) + m_3(\beta\xi)\} + G_1(\xi) \tag{80}$$

$$\frac{M_\xi}{K} = C_1(-2\beta^2)m_1(\beta\xi) + C_2\beta\{-m_2(\beta\xi) + m_3(\beta\xi)\} + G_3(\xi) \tag{81}$$

where

$$C_1 = \frac{-k_{22}g_1 + k_{12}g_2}{k_{11}k_{22} - k_{12}k_{21}}$$

$$C_2 = \frac{-k_{21}g_1 + k_{11}g_2}{k_{12}k_{21} - k_{11}k_{22}}$$

$$k_{11} = k_{22} = m_4(\beta H)$$

$$k_{12} = \frac{1}{2\beta}\left[m_2(\beta H) + m_3(\beta H)\right]$$

$$k_{21} = \beta\left[-m_2(\beta H) + m_3(\beta H)\right]$$

$$g_1 = \frac{W}{K}\frac{1}{4\beta^3}\{m_2[\beta(H-a)] - m_3[\beta(H-a)]\}$$

$$g_2 = \frac{W}{K}\frac{1}{2\beta^2}m_1[\beta(H-a)]$$

$$W = \frac{e}{H_h}P$$

$$m_1(\beta\xi) = \sin(\beta\xi)\sinh(\beta\xi)$$
$$m_2(\beta\xi) = \sin(\beta\xi)\cosh(\beta\xi)$$
$$m_3(\beta\xi) = \cos(\beta\xi)\sinh(\beta\xi)$$
$$m_4(\beta\xi) = \cos(\beta\xi)\cosh(\beta\xi)$$

$$0 \le \xi < a \quad G_1(\xi) = G_3(\xi) = 0$$

$$a \le \xi \le H \quad G_1(\xi) = \frac{W}{K}\frac{1}{4\beta^3}\{m_2[\beta(\xi-a)] - m_3[\beta(\xi-a)]\}$$

$$G_3(\xi) = \frac{W}{K}\frac{1}{2\beta}\{m_2[\beta(\xi-a)] + m_3[\beta(\xi-a)]\}$$

K is the flexural rigidity;

P is the vertical prestressing force.

M_{0e} is obtained as M_ξ, where $\xi = H$.

When vertical bending moment in areas other than the tank wall bottom is necessary, it may be approximated by Formula (82).

$$M_a = M_0 + M_{0f} \tag{82}$$

where

M_a is the corrected vertical bending moment;

M_0 is the vertical bending moment when the wall thickness is assumed to be constant at t;

M_{0f} is the vertical bending moment when hinge-supported wall bottom is loaded with bending moment ($M_{0f} - M_{0c}$) or ($M_e - M_{0e}$).

Note that this calculation does not consider vertical bending moment at the wall bottom, M_{0v}, generated by the effect of Poisson's ratio, v, due to vertical prestress. This may be calculated by Formula (83) and added to M_{0e}:

$$M_{0v} = \frac{vP}{2\beta^2 R} \tag{83}$$

where

M_{0v} is the vertical bending moment at the wall bottom generated by the effect of Poisson's ratio, ν, due to vertical prestress when the tank wall thickness is constant;

R is the radius of the tank wall.

11.5 Base slab

11.5.1 Structural types

The structural types of base slabs include as follows.

— One-layer base slab

A base slab shall be of a structure capable of resisting the deformation and subsidence of foundations with its rigidity and simultaneously retaining its watertightness. Since large stress is generated in the circumferential area where the deadweight of the tank wall and other loads are concentrated, this area is generally thickened and referred to as a ring plate.

— Two-layer base slab

In this type, the tank body is constructed on the foundation slab (the lower layer of the slab) that is constructed with reinforced or prestressed concrete on the foundation ground, in such a case as when uniform bearing capacity of the ground cannot be obtained. When the foundation ground is sufficiently firm as in the case of reliable bedrock, the foundation slab may not be constructed. The ring plate resists the stress in the base slab (the upper layer of the slab) due to interaction with the wall. A thin membrane floor is provided to ensure watertightness.

— Others

When the tank wall bottom is fixed-supported, the length of the ring plate may be determined using Formula (84).

$$\frac{M_{0c}}{2\beta K} = \left(\frac{H\rho}{4\beta K} + \frac{3H\rho}{2E_c t_{rp}^3} L_{rp} \right) L_{rp}^2 \tag{84}$$

where

M_{0c} is the vertical bending moment at the bottom of the tank wall;

β is the characteristic value of base slab;

K is the flexural rigidity;

ρ is the unit weight of water;

H is the total water depth;

E_c is the elastic modulus of concrete;

t_{rp} is the thickness of ring plate (see Figure 48);

L_{rp} is the length of ring plate (see Figure 48).

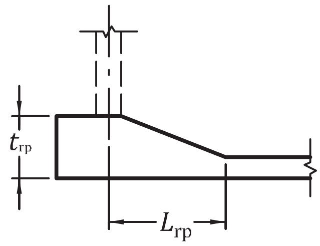


Figure 48 — Ring plate

11.5.2 Design in general

11.5.2.1 Loads for base slab

The loads for the design of a base slab shall generally include as follows:

— deadweight in accordance with 6.2;

— imposed load in accordance with 6.3;

— hydrostatic pressure;

— prestressing force;

— effect of an earthquake in accordance with 6.8 and Clause 9;

— snow load in accordance with 6.10;

— earth pressure in accordance with 6.11;

— uplift force in accordance with 6.12.

11.5.2.2 Load combinations

Combinations of loads for the design of a base slab shall generally be as given in Table 23. In addition, other loads that have strong effects on the base slab shall be considered as the need arises.

Table 23 — Load combinations

			Immediately after prestressing	Under ordinary conditions		During an earthquake
				Empty	Full	
Primary load	Deadweight		○	○	○	○
	Imposed load			○	○	○
	Hydrostatic pressure				○	○
	Prestressing force	Immediately after	○			
		Effective		○	○	○
Particular load	Snow load			○	○	○
	Earth pressure			○		

The deadweight, imposed loads and snow loads may be treated as vertical concentrated loads transferred from the tank wall bottom to the base slab. In regard to prestressing forces, hydrostatic pressure and earth pressure, the reaction forces generated from these loads at the tank wall bottom may be regarded as loads acting on the base slab. However, when horizontal reaction forces act as compressive forces, these may generally be neglected as being advantageous for the members. In such a case, only bending forces require consideration.

For a one-layer base slab, analysis may be carried out by applying the above-mentioned loads simultaneously to the joint between the tank wall and base slab. For a two-layer base slab, analysis may be carried out by simultaneously applying vertical concentrated load to the foundation slab at the joint between the tank wall and foundation slab and applying vertical concentrated load and bending moment to the base slab at the joint between the tank wall and base slab. In this case, the joint between the tank wall and base slab or foundation slab is defined as the point at which the vertical line passing the point of contact between the centroid of the tank wall and the upper surface of the base slab intersects with the centroid of the base slab or foundation slab.

11.5.2.3 General design of base slab

When designing a base slab, the following shall be observed:

— The thickness of a base slab at the joint with the tank wall shall be not less than the thickness of the tank wall bottom and not less than 0,3 m. At joints between the tank wall and base slab, rigidity equal to or greater than that of the tank wall shall be provided to the base slab to make the joints structurally stable. Also, in view of the congested reinforcement and prestressing steel, the thickness of the base slab shall be determined in consideration of the stress distribution in the area embedding anchorages for vertical prestressing steel. The concrete strength of the ring plate shall be not less than f'_{ck} = 30 MPa when no particular investigation is conducted for anchorages. However, the concrete strength can be reduced subject to sufficient investigation.

— In a pile-foundation tank, joints between the base slab and piles shall be fixed or hinged connections and pile heads shall be appropriately treated. When selecting the connection method at pile heads, care shall be exercised regarding the type of the structure, function of the structure, shape and dimensions of the base slab or foundation slab, type of piles, ground conditions and difficulty level of connecting work.

  When hinged connection is adopted, it is generally difficult to expect a perfect hinge. Investigation shall therefore be conducted by applying the maximum bending moment based on hinge calculation to pile heads. When hinged connection is adopted for prestressed concrete piles with heads cut off, particular care shall be exercised for the strengthening of pile head zones. When pile heads are hinge-connected, no pull-out force should as a rule be generated in the piles.

  The effects of pile head displacement, which are generally conceivable during an earthquake, shall be considered to prevent deleterious effects on piping, fixtures and fittings.

— For pile foundations, the base slab and foundation slab may be assumed to be a flat slab or continuous beam on elastic supports for calculation. However, the slab thickness is required to be not less than the pile diameter in consideration of the rigidity of piles and the slab. The stress limit for reinforcement on the side in contact with water is determined in consideration of watertightness.

11.5.2.4 One-layer base slab structure

When designing a one-layer base slab, the following shall be observed in addition to 11.5.2.3:

— Analysis shall be carried out in consideration of the characteristics of the ground and rigidity of the base slab,

— The minimum thickness of the disk of the base slab shall be 0,16 m,

— The minimum reinforcement in one direction of the base slab shall be the cross-sectional area multiplied by the values given in Table 24 when ribbed bars are used. When plain bars are used, the values in the table shall be increased by 0,05 %.

Table 24 — Minimum reinforcement in one direction of a base slab

Thickness, t_s (m)	**≤0,15**	**0,15 – 1,0**	**≥ 1,0**
Minimum reinforcement ratio (%)	0,45	$0{,}45-(t_s-0{,}15)\dfrac{0{,}2}{0{,}85}$	0,25

In the case of a spread foundation, the base slab should be analysed as a disk on an elastic floor. In this case, it is advisable to apply water pressure as loads distributed from the inside of the tank wall inward for analysis, in consideration of extension outward beyond the tank wall line.

A partially loaded disk with changing cross-sections on an elastic floor can be analytically dissolved as combinations of rings on an elastic floor. However, the analysis results, which are obtained as series solutions, are quite complicated. The analysis may therefore be carried out by FEM using axisymmetric thin wall shell elements or a planar frame model. The planar frame model is a sectoral element of 1 rad from the disc. The vertical springs representing ground springs should normally be distributed springs, concentrated springs are assumed in this analysis. Rotational springs, which are determined depending on the circumferential flexural rigidity, are intended to satisfy the circumferential continuity that was lost by cutting of the element. Note that consideration of the in-plane forces (radial axial forces) of the disc further requires horizontal springs determined by circumferential axial rigidity. However, it is not necessary in the case where the axial line is horizontal and vertical loads and concentrated moment loads are dealt with.

This model provides a sort of approximate solution because the concentrated springs are assumed, but it is not necessary to increase the divisions to increase the accuracy.

Section moduli and spring constants necessary for this planar frame (beam) model are approximately determined as Formula (85):

$$K_{vi}=\frac{K_v}{8}\left[\left(r_{i+1}+r_i\right)^2-\left(r_i+r_{i-1}\right)^2\right]$$

$$K_{\theta i}=\frac{E}{96\left(1-\upsilon^2\right)}\left[\left(t_{i-1}+t_i\right)^3\lg\frac{2r_i}{r_{i-1}+r_i}+\left(t_i+t_{i+1}\right)^3\lg\frac{r_i+r_{i+1}}{2r_i}\right] \tag{85}$$

$$A_i=\frac{r_i+r_{i+1}}{2}\frac{t_i+t_{i+1}}{2}$$

$$I_i=\frac{r_i+r_{i+1}}{192}\left(t_{i+1}+t_i\right)^3$$

where

K_{vi}, $K_{\theta i}$	are the vertical and rotational spring constants, respectively, of node i;
A_i, I_i	are the cross-sectional area and second moment of area, respectively, of element i (member between nodes i and i + 1);
r_i	is the radius of node i (X coordinate);
t_i	is the thickness of base slab at node i;
E	is the elastic modulus;
K_v	is the vertical subgrade reaction modulus.

Since concentrated springs at nodes are assumed in the above-mentioned model, it is advisable that distributed loads also be assumed as concentrated loads at nodes. The load at a node is determined as follows:

— Concentrated load

$$P_{\mathrm{F}i} = P_{\mathrm{s}i} r_i \tag{86}$$

— Water pressure (distributed load)

$$P_{\mathrm{F}i} = \frac{\rho L_{\mathrm{w}}}{8}\left[\left(r_{i+1} + r_i\right)^2 - \left(r_i + r_{i-1}\right)^2\right] \tag{87}$$

where

$P_{\mathrm{r}i}$ is the concentrated load acting on node i (including moment load);

$P_{\mathrm{s}i}$ is the concentrated load per unit length acting on node i;

ρ is the unit weight of water;

L_{w} is the depth of water.

The results of analysis using the above-mentioned data can be converted to bending moment per unit length as Formula (88):

$$\begin{aligned} M_{\mathrm{r}i} &= \frac{M_{\mathrm{x}i}}{r_i} \\ M_{\theta i} &= \upsilon M_{\mathrm{r}i} + \frac{D_i}{r_i}\left(1 - \upsilon^2\right)\phi_i \end{aligned} \tag{88}$$

where

$M_{\mathrm{r}i}$, $M_{\theta i}$ are the bending moment per unit length in the radial and circumferential directions at node i, respectively;

$M_{\mathrm{x}i}$, ϕ_i are the bending moment and deflection angle, respectively, at node i determined by planar frame analysis;

D_i is the flexural rigidity at node i $\left[= \dfrac{Et_i^3}{12\left(1 - \nu^2\right)}\right]$.

When the thickness is constant or when the thickness changes are negligibly small, analysis may be carried out by assuming a beam on an elastic floor instead of a disc on an elastic floor. However, in this case as well, the loads transferred from the tank wall should be assumed as concentrated loads on the joint with the tank wall and the water pressure should be assumed as distributed loads from the inside of the tank wall inward, in consideration of the extension beyond the tank wall line, similarly to the case of a disc. When analysis is conducted assuming a beam, the circumferential moment is not obtained. Therefore, the same amount of reinforcement as in the radial direction should be provided in the circumferential direction. For this reason, it is desirable to carry out analysis assuming a disc when the circumferential reinforcement content affects the economic efficiency, such as for a tank with a large capacity.

11.5.2.5 Two-layer base slab structure

When designing a base slab of this type, the following shall be observed in addition to 11.5.2.3.

— No bending moment shall be borne in the membrane floor. The membrane floor is required to bear no bending moment by reducing its thickness and reducing its rigidity ratio to the foundation slab, etc.

— The foundation slab shall be analysed in consideration of the properties of the ground and the rigidity of the slab.

— The minimum thickness of the membrane floor shall be [0,1] m.

— The minimum thickness of the foundation slab shall be [0,2] m.

— The minimum reinforcement in one direction of the base slab shall be the cross-sectional area multiplied by the values given in Table 24.

— The minimum reinforcement when using ribbed bars in one direction shall be the cross-sectional area multiplied by 0,25 %. When using plain bars, the values shall be increased by 0,05 %.

12 Materials

12.1 Quality of materials

12.1.1 General

Materials to be used for a PC water tank shall be of qualities that comfortably meet the requirements for the purpose of the tank. Basically, all materials shall be used with confirmation to the relevant ISO standards.

NOTE Examples of materials specifications are given in Annex D.

12.1.2 Concrete materials

12.1.2.1 Cement

Cement that provides watertight concrete shall be selected for a PC water tank with thorough consideration to the time and methods of construction and climatic conditions of the construction site.

12.1.2.2 Water

Mixing water shall be conformed to ISO 12439.

12.1.2.3 Fine aggregate

Fine aggregate shall be clean, hard and durable with an adequate grading. It shall not contain a harmful amount of dirt, mud, organic impurities, chlorides, or the like.

12.1.2.4 Coarse aggregate

Coarse aggregate shall be clean, hard and durable with an adequate grading. It shall not contain a harmful amount of thin stone chips, long stone chips, organic impurities, chlorides, or the like. When fire resistance is particularly required, fire-resistant coarse aggregate shall be used.

12.1.2.5 Admixtures

Supplementary cementitious materials and chemical admixtures used as admixtures shall be of confirmed quality.

12.1.3 Concrete

12.1.3.1 General

Concrete shall be highly watertight and durable with the required strength.

12.1.3.2 Strength

The strength of concrete shall be specified by test values of standard-cured specimens at an age of 28 days.

The compressive strength, tensile strength and flexural strength of concrete shall be tested in accordance with ISO 1920-4. Specimens for these tests shall be fabricated in accordance with ISO 1920-3.

12.1.4 Prestressing steel

Prestressing steel conforming to ISO 6934-1, ISO 6934-2, ISO 6934-3, ISO 6934-4 or ISO 6934-5 shall be used.

When using prestressing steel other than the above-mentioned types, its qualities shall be confirmed by tests and adequate strength and other design values shall be separately established.

Also, when prestressing steel is to be reworked or heat-treated for anchorage, connection, assembly, or placing, tests shall be conducted beforehand to confirm that such rework or treatment would not impair the qualities of prestressing steel.

12.1.5 Steel reinforcement

Steel reinforcement to be used shall conform to ISO 6935-1, ISO 6935-2 or ISO 6935-3; otherwise, tests shall be conducted to establish the design values and method of use.

Epoxy-coated steel reinforcement to be used shall conform to ISO 14654.

12.1.6 Welded wire fabric

Welded wire fabric to be used as reinforcement for concrete shall have the required strength and shall be suitable for the construction conditions.

Welded wire fabric to be used shall conform to ISO 6935-3.

Welded wire fabric as reinforcement for concrete is often used for a dome and a base slab. However, sufficient care should be exercised for anchorage when using welded wire fabric for these members. The shapes and dimensions of the mesh and the diameter of steel wires should be appropriately selected according to the purpose of the structure.

12.1.7 Anchorages and couplers

Anchorages and couplers shall have such structure and strength that they would not be broken or significantly deformed before the anchored or connected prestressing steel achieves the tensile load required by the standard. The performance of anchorages and couplers shall as a rule be confirmed based on relevant ISO standards.

12.1.8 Sheath

Sheaths shall not be readily deformed during handling or concrete placing. Also, their structure shall be designed to prevent cement paste from entering through their joints.

The shapes and dimensions of sheaths shall be determined in consideration of ease of inserting prestressing steel, ease of filling prestressing grout, their capability of ensuring the bond and friction with prestressing steel.

12.1.9 Coating materials for protecting prestressing steel

12.1.9.1 Grout for prestressed concrete

Grout shall be capable of completely filling the inside of the ducts to encase prestressing steel, so as to protect them from corrosion, while providing monolithic bond between prestressing steel and member concrete.

The qualities of grout shall meet the following requirements by testing in accordance with ISO 14824-3:

— consistency;

— expansion ratio;

— bleeding ratio;

— strength;

— chloride content limit;

Other recommendations as follows shall be referred:

Water to be used for grout shall not contain a harmful amount of any substance having an adverse effect on the grout or prestressing steel.

Investigation shall be made beforehand as to whether or not admixtures in grouts may be used, their qualities and how they should be used.

The water-cement ratio of grout shall be not more than 45 %.

12.1.9.2 Coating materials for unbonded prestressing steel

Coating materials for unbonded prestressing steel shall be such that they do not cause corrosion on prestressing steel, damage concrete, or cause no bond between prestressing steel and concrete during prestressing.

13 Tank appurtenances

13.1 Ladders/stairs and handrails

A tank shall be provided with means of climbing up or down for access to the inside of the tank. Also, sturdy handrails with a height of not less than 0,75 m shall be provided for the area leading to hatches.

Means of climbing up or down should generally be stairs or ladders. For the inside, ladders are used in most cases. Since ladders on the inside are to be immersed in water, those made of stainless steel or other rustproof materials are preferable.

Stairs shall have a width of not less than [0,6] m. For a stair with a height exceeding [8] m, it is preferable to provide a landing for each height of less than [7] m.

The width of a ladder shall be not less than [0,3] m. The steps of a ladder should be equally spaced at approximately [0,25] m. The uppermost step of an external ladder shall protrude not less than [0,6] m from the floor. A basket or landing shall be provided for an external ladder with a particularly large height.

13.2 Manhole and water pilot hole

A PC water tank shall be provided with manhole and water pilot hole. Manhole and sample water pilot hole shall be constructed so as to prevent rainwater ingress and shall be locked.

13.3 Ventilators

Ventilators shall be provided in a PC water tank.

Ventilators shall meet the following requirements.

— The ventilators shall have a ventilation area to pass air at a flow rate equal to the maximum tank outflow rate per day. The number of ventilators shall be as small as possible.

— The ventilators shall be configured to prevent entry of rainwater, dust and pests/insects from outside.

13.4 Lightning rods

A lightning rod shall be installed for a water tank with a height exceeding [20] m from the ground. It is advisable to install a lightning rod on a lower tank when it is particularly prone to lightning strikes.

A lightning rod comprises an aerial terminal, cables and an earth electrode. The protection range is within 60° of the tip of the rod.

13.5 Piping

The inlet and outlet piping of a PC water tank shall meet the following requirements.

— The numbers and locations of inlet and outlet piping shall be determined in consideration of the shape and configuration of the tank so that the tank water may not stop flowing.

— The level of the cross-sectional centre of the outlet pipe shall be lower than the low water level by not less than two times the pipe diameter.

— Care shall be exercised for watertightness where a pipe passes through the tank wall or base slab. Flexible joints shall be provided near the outside of the wall as necessary.

— Cutoff valves shall be provided for both the inlet and outlet pipes.

— It is advisable to provide an emergency shutoff valve for the outlet pipe as necessary.

— The inlet pipe shall be provided with either an overflow weir or check valve, or the pipeline shall be arranged over the tank wall.

The overflow pipe shall meet the following requirements.

— The overflow pipe shall be provided at the high water level and shall have a trumpet-shaped opening or weir.

— The tank overflow shall be determined in consideration of the area of the tank, clearance and inflow rate.

— The high water level at the discharge end of the overflow pipe shall be lower than the high water level of the tank.

The drainage pipe shall meet the following requirements.

— A drainage pipe having a shutoff valve shall be provided at the low point of the pond.

— The diameter of the drainage pipe shall be determined in consideration of the amount of water below the low water level.

— It is desirable that the high water level at the discharge end of the drainage pipe be lower than the low point of the tank. However, where the whole amount of water cannot be drained by gravity, a pump drainage system including a catch basin shall be provided.

The tank bypass shall meet the following requirements.

— A bypass shall be provided to allow water conveyance without passing through the tank.

— A check valve shall be provided in the bypass.

13.6 Catch basin

When planning a catch basin at the bottom of a PC water tank, points of high stress shall be avoided to the extent that it is possible.

It is desirable that one side of a catch basin is not shorter than three times the diameter of the outlet pipe.

13.7 Water-level gauge

A water-level gauge shall be provided for a PC water tank to permit remote level indication. The level indicator shall be connected to an alarm system as necessary.

13.8 Rainwater treatment

The roof of a PC water tank shall be provided with drains and gutters to allow rainwater to flow ofE.

13.9 Protection equipment

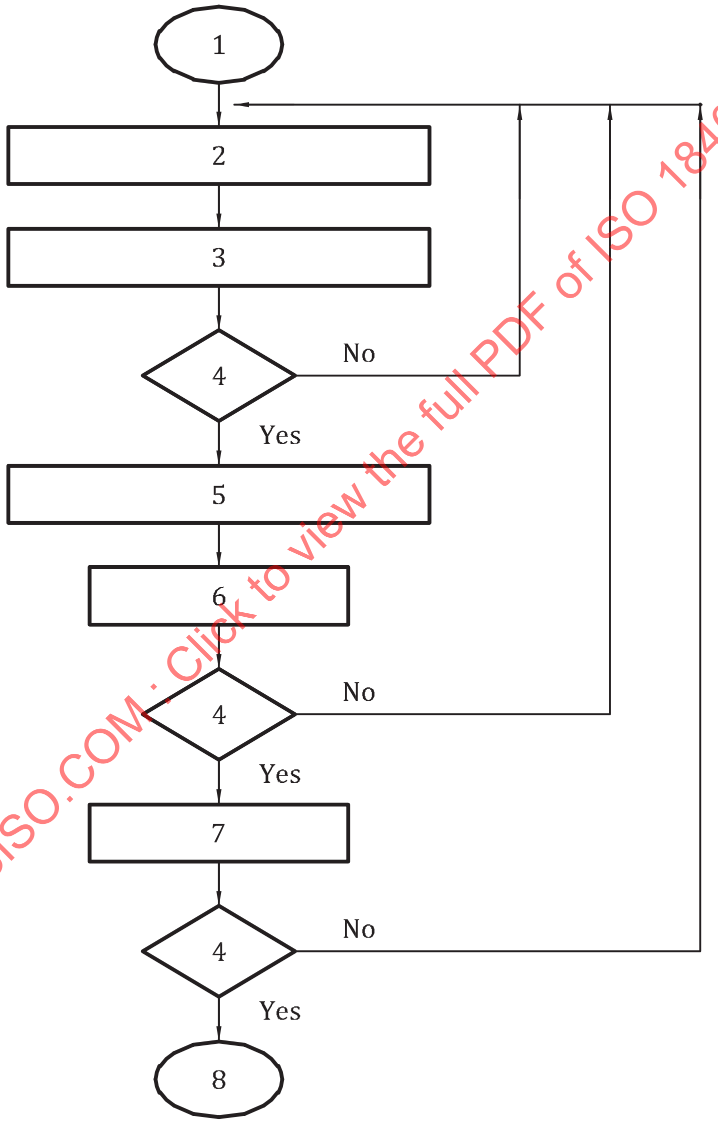
Protection equipment shall be provided to prevent unauthorized entry, which can cause accidents. Entrances to stairs, ladders, and corridors should be fenced in and locked.

The entire tank site should also be fenced in, and the entrances should be locked.

Annex A
(informative)

Reference design flow

The design calculation flow in according with this document is shown as Figure A.1. Additional explanation with reference sections is given in Table A.1.



Key

1 start
2 confirmation of design conditions
3 calculation in ordinary condition
4 check
5 verification of earthquake conditions
6 Level 1 earthquake
7 Level 2 earthquake
8 end

Figure A.1 — Design calculation flow

Table A.1 — Explanation of design flow

No	Design item: Matter	Design item: Detail	Design investigation: Confirming states	Design investigation: Remark positions	Design investigation: Check conditions	Reference clause in this document	
1	Confirmation of design condition	Kinds of structures	—	—	—	5	11
		Shapes				11	
		Materials				7	12
		Analysis methods				7	11
		Loads				6	
		Seismic conditions				9	
		Soil and foundation				11	
		Others				11	
2	Design calculation in ordinary conditions	Primary load	Watertightness	All	Stress limits	6	
		Subsidiary load				8	
		Particular load				11	
3	Verification of earthquakes conditions	Level 1	No damage	All	Stress limits	8	
		(conventional earthquakes)				9	
		Level 2 (very strong earthquakes)	Retainable function safety	Circumference wall	Bearing capacity and response strain	9	9.3.4
				Vertical wall	Bearing capacity		
		Foundation	Stability and safety	Spread foundation	Stability (Level 1)	9	9.4
				Pile foundation	Limit state (Levels 1 and 2)		
4	Confirmation of general structural details	—	—	—	—	10	
5	Reference of appurtenances	—	—	—	—	13	

Annex B
(informative)

Design seismic coefficients for the seismic coefficient method

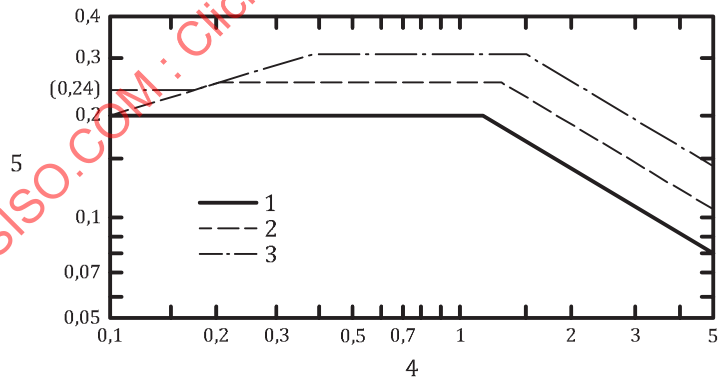
B.1 Level 1 ground motion

To determine the seismic coefficient for verification of safety against earthquakes, a level of seismic hazard should be defined for a PC water tank in terms of the intensity of the effective peak ground horizontal acceleration in bedrock at the tank site. For example, according to the analysis on strong motion seismograms of 394 components observed in Japan, standard horizontal seismic coefficients for Level 1 ground motion are given in Table B.1 and illustrated in Figure B.1.

Table B.1 — Standard horizontal seismic coefficients (Level 1 ground motion)

Ground type	Value of K_{h01} for natural period of structure, T (s)		
Type I ground (T_G < 0,2)	T < 0,1 $K_{h01} = 0{,}431T^{1/3} \geq 0{,}16$	$0{,}1 \leq T \leq 1{,}1$ $K_{h01} = 0{,}2$	$1{,}1 < T$ $K_{h01} = 0{,}213T^{-2/3}$
Type II ground [$0{,}2 \leq T_G < 0{,}6$]	T < 0,2 $K_{h01} = 0{,}427T^{1/3} \geq 0{,}20$	$0{,}2 \leq T \leq 1{,}3$ $K_{h01} = 0{,}25$	$1{,}3 < T$ $K_{h01} = 0{,}298T^{-2/3}$
Type III ground ($0{,}6 \leq T_G$)	T < 0,34 $K_{h01} = 0{,}430T^{1/3} \geq 0{,}24$	$0{,}34 \leq T \leq 1{,}5$ $K_{h01} = 0{,}3$	$1{,}5 < T$ $K_{h01} = 0{,}393T^{-2/3}$

NOTE T_G is the natural period of the ground (s).



Key
1 type I ground
2 type II ground
3 type III ground
4 natural period, T (s)
5 standard horizontal seismic coefficient of ground motion, K_{h01}

Figure B.1 — Standard horizontal seismic coefficient (K_{h01})

As for the scope of this document, the correction factor, C_z, defined in Formula (27) should be taken as specified in Table B.2. The seismic hazard is divided in three different zones in Japan as shown in Figure B.2, regions A, B and C, for example. The global seismic hazard is given in ISO 15673.

Table B.2 — Regional correction factors for Japan

Region division	Correction factor, C_z
Region A	1,0
Region B	0,85
Region C	0,7

The ground type for seismic design is classified based on the natural period of the ground, T_G, calculated from Formula (B.1). If the ground surface coincides with the rock surface, the ground is classified as Type I.

$$T_G = 4\sum_{i=1}^{n}\frac{H_i}{V_{si}} \tag{B.1}$$

where

T_G is the natural period of the ground;

H_i is the thickness of the i-th stratum;

V_{si} is the mean shear elastic wave velocity of the i-th stratum.

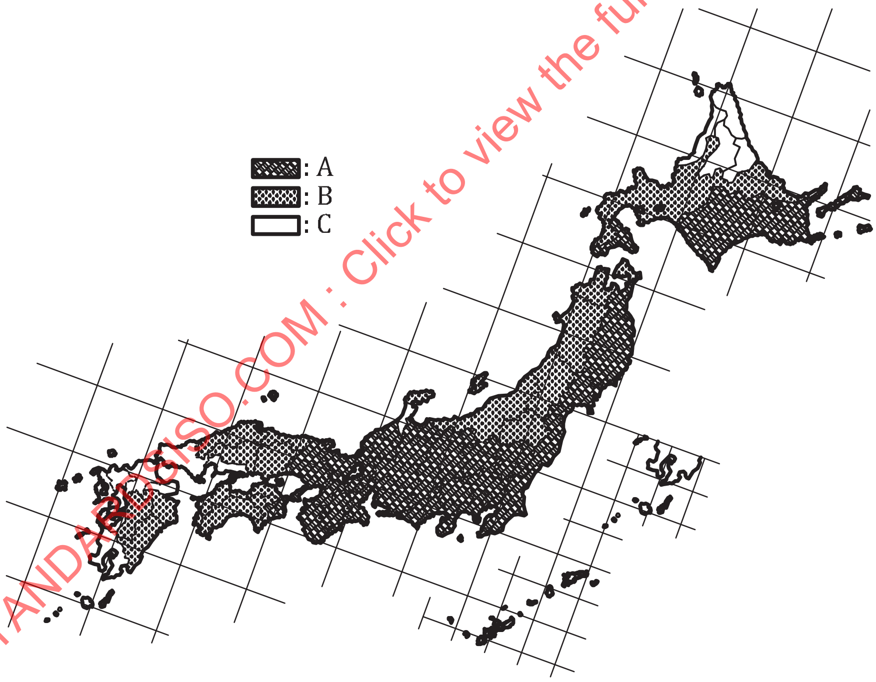


Figure B.2 — Regional division for Japan

The ground types are used for determining the design horizontal seismic coefficient (K_{h1}) for the seismic coefficient method. Type I ground may be regarded as good diluvial ground and bedrock; Type III, weak ground among alluvial ground; and Type II, diluvial and alluvial grounds not included in Types I and III. Alluvial strata referred to here include new sedimentary layers due to cliff failure, etc., topsoil, reclaimed soil and weak layers. Diluvial strata include tight sand layers, gravel layers and cobble layers among alluvial layers. Though it is desirable to measure V_S by elastic wave prospecting or PS logging, it may be estimated from the N value (N) according to Table B.3 when no measurement data are available. The N value should represent the average of the N values (obtained with the standard penetration test) of the component layers. It is not necessary to overcomplicate this calculation.

Table B.3 — Shear elastic wave velocity of grounds

Soil classification		V_S (m/s)		
		10^{-3}	**10^{-4}**	**10^{-6}**
Diluvial	Cohesive soil	129 $N^{0,183}$	156 $N^{0,183}$	172 $N^{0,183}$
	Sandy soil	123 $N^{0,125}$	200 $N^{0,125}$	205 $N^{0,125}$
Alluvial	Cohesive soil	122 $N^{0,077\,7}$	142 $N^{0,077\,7}$	143 $N^{0,077\,7}$
	Sandy soil	61,8 $N^{0,211}$	90 $N^{0,211}$	103 $N^{0,211}$

NOTE 1 These were classified by the percent fractions of sand and clay components.

NOTE 2 Use shear strains of a 10^{-3} level for the surface layer. For foundations, use the values of a 10^{-6} level.

B.2 Level 2 ground motion

The standard horizontal seismic coefficient for Level 2 ground motion is defined at the position of the centroid of the structure as specified for each ground type. The standard horizontal seismic coefficients for Level 2 ground motion are listed in Table B.4 for reference.

Table B.4 — Standard horizontal seismic coefficients (Level 2 ground motion)

Ground type	Value of K_{h02} for natural period of structure, T (s)		
Type I ground ($T_G < 0,2$)	$T < 0,2$ $K_{h02} = 2,291T^{0,515} \geq 0,70$	$0,2 \leq T \leq 1,0$ $K_{h02} = 1,0$	$1,0 < T$ $K_{h02} = 1,000T^{-1,465}$
Type II ground [$0,2 \leq T_G < 0,6$]	$T < 0,2$ $K_{h02} = 5,130T^{0,807} \geq 0,80$	$0,2 \leq T \leq 1,0$ $K_{h02} = 1,4$	$1,0 < T$ $K_{h02} = 1,400T^{-1,402}$
Type III ground ($0,6 \leq T_G$)	$T < 0,3$ $K_{h02} = 2,565T^{0,631} \geq 0,60$	$0,3 \leq T \leq 1,5$ $K_{h02} = 1,2$	$1,5 < T$ $K_{h02} = 2,003T^{-1,263}$

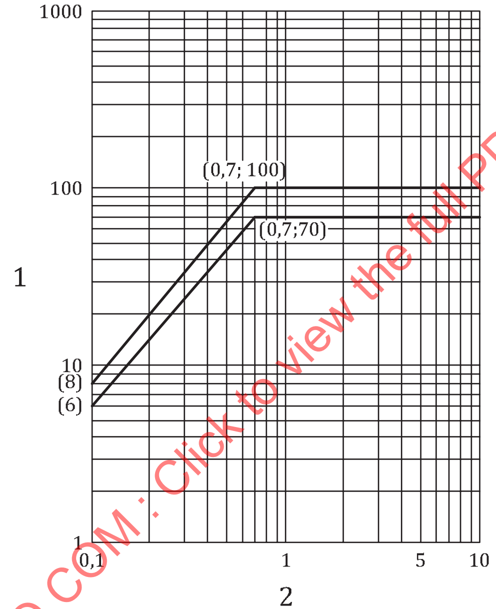
NOTE T_G is the natural period of the ground (s).

Annex C
(informative)

Seismic input for design by dynamic analysis

When using dynamic analysis to verify for the safety against Level 2 ground motion, the following seismic waves may be used: seismic waves conforming to the velocity response spectra of bedrock and the acceleration response spectra of surface ground or actual seismic waves observed near inland faults at site.

For velocity response spectra of bedrock and the acceleration response spectra of surface ground are shown in Figure C.1 and Figure C.2, respectively. Two lines in each figure correspond to 90 % and 70 % of non-excess probability.

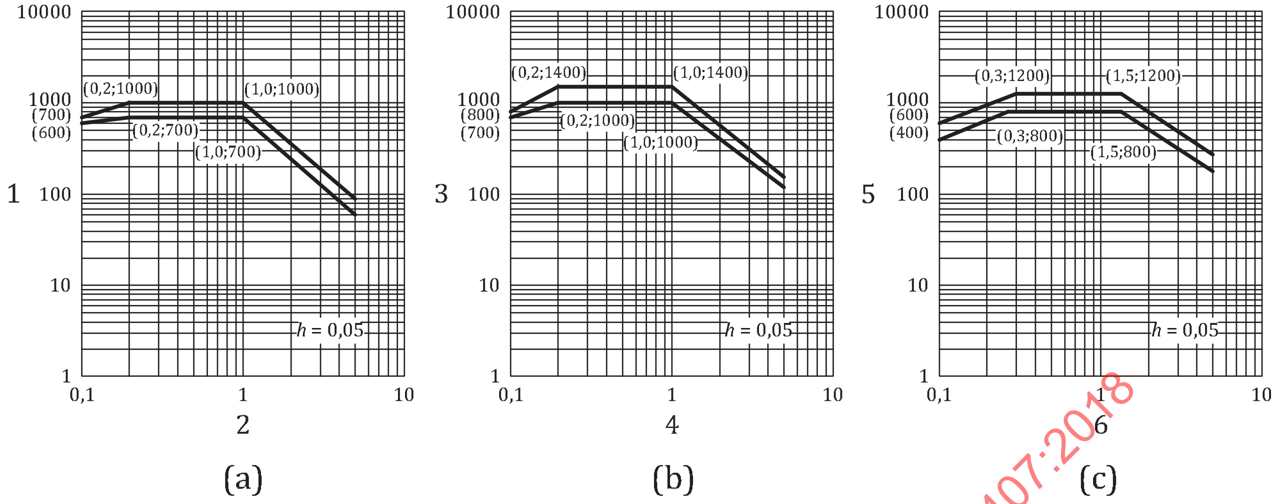


Key

1 velocity response spectrum, S_V (cm/s)

2 natural period, T_G (s)

Figure C.1 — Velocity response spectra for design regarding Level 2 ground motion



Key

1 acceleration response spectra (gal)
2 natural period (s)
3 acceleration response spectra (gal)
4 natural period (s)
5 acceleration response spectra (gal)
6 natural period (s)

a) Type I ground ($T_G < 0{,}2$ s)
b) Type II ground ($0{,}2\text{ s} \leq T_G < 0{,}6$ s)
c) Type III ground ($0{,}6\text{ s} \leq T_G$)

Figure C.2 — Acceleration response spectra regarding Level 2 ground motion

When selecting observed seismic waves to be used for dynamic analysis regarding Level 2 ground motion, the ground type of the observation points should be considered and the similarity of the response spectra of the observed seismic waves to those response spectra for design shown in Figure C.2 should be confirmed. Also, the maximum values of input seismic waves for dynamic analysis should be 600 gal to 700 gal, 700 gal to 800 gal and 400 gal to 600 gal for Type I, Type II and Type III grounds, respectively, in the surface ground, and 400 gal to 500 gal for bedrock.

Annex D
(informative)

Example of material specifications

D.1 Concrete

D.1.1 Modulus of elasticity

The values listed in Table D.1 may be used as the elastic modulus for the calculation of indeterminate forces or elastic deformation of a reinforced concrete structure and the design calculation of prestressed concrete members.

Table D.1 — Elastic modulus of concrete

	Characteristic compressive strength, f'_{ck} (MPa)					
	21	**24**	**30**	**40**	**50**	**60**
E_c (GPa)	23,5	25	28	31	33	35

D.1.2 Drying shrinkage

The degree of drying shrinkage of concrete to be used for calculating the prestress loss may be assumed to be the values given in Table D.2.

Table D.2 — Drying shrinkage of concrete

	Age of concrete at prestressing (day)			
	4–7	**28**	**90**	**365**
Shrinkage strain	20×10^{-5}	18×10^{-5}	16×10^{-5}	12×10^{-5}

D.1.3 Creep

The creep factor for calculating the prestress loss and indeterminate force may be assumed to be the values given in Table D.3.

Table D.3 — Creep factor of concrete

		Age of concrete when a sustained load is applied (day)				
		4–7	**14**	**28**	**90**	**365**
Creep factor	With high early strength Portland cement	2,6	2,3	2,0	1,7	1,2
	With ordinary Portland cement	2,8	2,5	2,2	1,9	1,4

D.2 Steel

D.2.1 Modulus of elasticity

The elastic modulus of steel for design calculation may be assumed to be the values given in Table D.4.

Table D.4 — Elastic modulus of steel

Steel type	Elastic modulus (GPa)
Reinforcing bar	210
Prestressing steel	200

D.2.2 Relaxation

As to the apparent relaxation ratio of prestressing steel for calculating the prestress loss, the ratios may be assumed to be the values given in Table D.5.

Table D.5 — Apparent relaxation ratio of prestressing steel, γ

Type of prestressing steel	Apparent relaxation ratio
Prestressing wire and prestressing strands	5 %
Prestressing bars	3 %
Low-relaxation prestressing steel	1,5 %

D.3 Stress limit

D.3.1 Stress limit of reinforced concrete members

D.3.1.1 Stress limit of concrete

The compressive stress and shear stress limits of concrete may be assumed to be the values given in Table D.6.

Table D.6 — Compressive and shear stress limits

Dimensions in MPa

Stress type			Design strength of concrete				
			21	24	27	30	≥40
Compressive stress	Flexural compressive stress		8,0	9,0	10,0	11,0	14,0
	Axial compressive stress		5,5	6,5	7,5	8,5	11,0
Shear stress	Without calculation for diagonal bars	Beam	0,42	0,45	0,47	0,50	0,55
		Slab	0,85	0,9	0,95	1,0	1,1
	With calculation for diagonal bars		1,9	2,0	2,1	2,2	2,4

The bond stress limit of concrete with reinforcement with a diameter of not more than 32 mm may be assumed to be the values given in Table D.7

Table D.7 — Bond stress limit of concrete

Dimensions in MPa

Rebar type	Design strength of concrete				
	21	24	27	30	≥40
Plain bar	0,75	0,80	0,85	0,90	1,00
Ribbed bar	1,5	1,6	1,7	1,8	2,0

D.3.1.2 Stress limit of reinforcement

The stress limit of reinforcement with a diameter of not more than 32 mm may be assumed to be the values given in Table D.8.

Table D.8 — Stress limit of reinforcement

Dimensions in MPa

Specified characteristic value of upper yield strength		235	295	345
Immediately after prestressing	a) General member	171	220	245
	b) Member in contact with water	122	122	122
	c) When calculating the length of lap splices or development length of rebars	171	220	245
	d) Compressive stress	171	220	245
Other	e) General member	137	176	196
	f) Member in contact with water	98	98	98
	g) When calculating the length of lap splices or development length of rebars	137	176	196
	h) Compressive stress	137	176	196

D.3.2 Stress limit of prestressed concrete members

D.3.2.1 Stress limit of concrete

The compressive stress limit of concrete may be assumed to be the values given in Table D.9.

Table D.9 — Compressive stress limit of concrete

Dimensions in MPa

Design strength of concrete		30	40	50
Immediately after prestressing	a) Flexural compressive stress	15	19	21
	b) Axial compressive stress	11	14,5	16
Other	c) Flexural compressive stress	12	15	17
	d) Axial compressive stress	8,5	11	13,5

The tensile stress limit of concrete may be assumed to be the values given in Table D.10.

Table D.10 — Tensile stress limit of concrete

Dimensions in MPa

Design strength of concrete		30	40	50
Flexural tensile stress	a) Immediately after prestressing	1,2	1,5	1,8
	b) Primary load and particular load equivalent to primary load (excluding hydrostatic pressure)	0,6	0,8	1,0
	c) Primary load and particular load equivalent to primary load	0	0	0
Axial tensile stress	d) Immediately after prestressing	0	0	0
	e) Primary load and particular load equivalent to primary load (excluding hydrostatic pressure)	0	0	0
	f) Primary load and particular load equivalent to primary load	0	0	0

The shear and diagonal tensile stress limits may be assumed to be the values given in Table D.11.

Table D.11 — Shear and diagonal tensile stress limits of concrete

Dimensions in MPa

Design strength of concrete	30	40	50
a) Shear stress	0,45	0,55	0,65
b) Axial tensile stress	0,8	0,9	1,0

The bond stress limit of concrete may be assumed to be the values given in Table D.12.

Table D.12 — Bond stress limit of concrete

Dimensions in MPa

Design strength of concrete	30	40	50
a) Plain bar	0,9	1,0	1,0
b) Ribbed bar	1,8	2,0	2,0

D.3.2.2 Tensile stress limit of prestressing steel

The tensile stress limit of prestressing steel may be assumed to be the values given in Table D.13.

Table D.13 — Tensile stress limit of prestressing steel

Stress condition	Tensile stress limit	Notes
a) During prestressing	0,80 f_{pu} or 0,90 f_{py}, whichever is smaller	f_{pu}: tensile strength of prestressing steel f_{py}: yield strength of prestressing steel
b) Immediately after prestressing	0,70 f_{pu} or 0,85 f_{py}, whichever is smaller	
c) While in service	0,60 f_{pu} or 0,75 f_{py}, whichever is smaller	

D.3.2.3 Stress limit of reinforcement

The stress limit of reinforcement may be as specified in D.3.1.2.

D.3.2.4 Augmentation of tensile stress limit of concrete

The tensile stress limit of concrete considering a subsidiary load may be assumed to be the values given in Table D.14, in which the flexural and axial tensile stress limits are combined as the tensile stress limit with consideration of a PC water tank.

Table D.14 — Tensile stress limit of concrete

Dimensions in MPa

Design strength of concrete Load combination	30	40	50
a) Primary load + particular load equivalent to primary load + effect of temperature (temperature difference)	1,7	2,0	2,3
b) Primary load + particular load equivalent to primary load + wind load	2,2	2,5	2,8
c) Primary load + particular load equivalent to primary load + effect of an earthquake	2,2	2,5	2,8

Annex E
(informative)

Example of design calculation

E.1 Outline

E.1.1 Outline of design

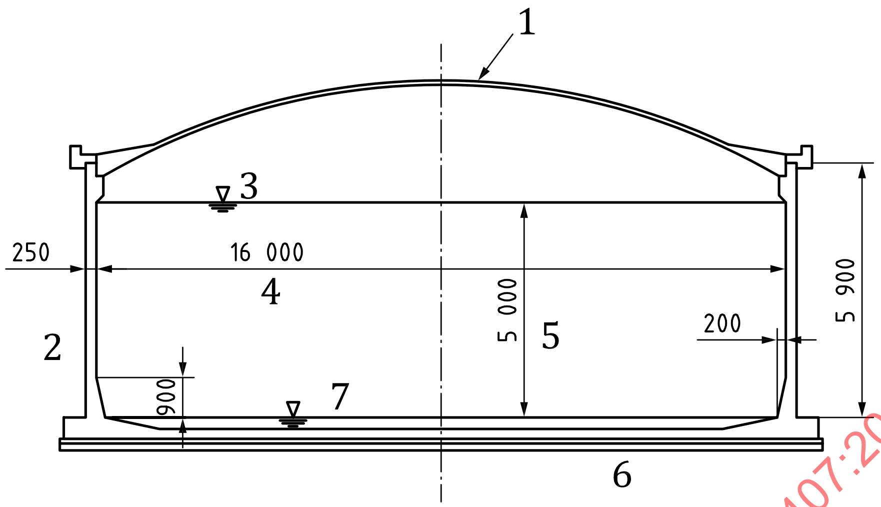
This annex provides an example of the design procedure of a standard-shaped prestressed concrete water tank (hereafter referred to as the PC water tank). The dome roof and base slab are made of reinforced concrete, whereas its wall and dome ring are of prestressed concrete construction. The design flow for the example is the same as that given in Annex A.

E.1.2 Outline of the structure

The PC water tank is a cylindrical prestressed concrete tank for potable water. The structural type of each member is given in Table E.1 and the cross-sectional view of the PC water tank is shown in Figure E.1.

Table E.1 — Structural type

Member	Structure	RC/PC
Dome roof	Statistically determined spherical shell	RC
Dome ring	Ring beam	PC
Wall	Cylindrical shell fixed at the bottom of the wall	PC
Base slab	One-layer circular base slab	RC
Foundation type	Spread foundations	—
NOTE PC and RC are prestressed concrete and reinforced concrete, respectively.		



Key

1 dome roof
2 wall
3 H. W. L.
4 inside diameter
5 calculated water depth
6 base slab
7 L. W. L.

Figure E.1 — Cross-section of the PC water tank

E.2 Design conditions

E.2.1 Basic dimensions and general shape

The basic dimensions of the PC wall tank are as follows:

Inside diameter, D = 16 m

Calculated water depth, H = 5 m

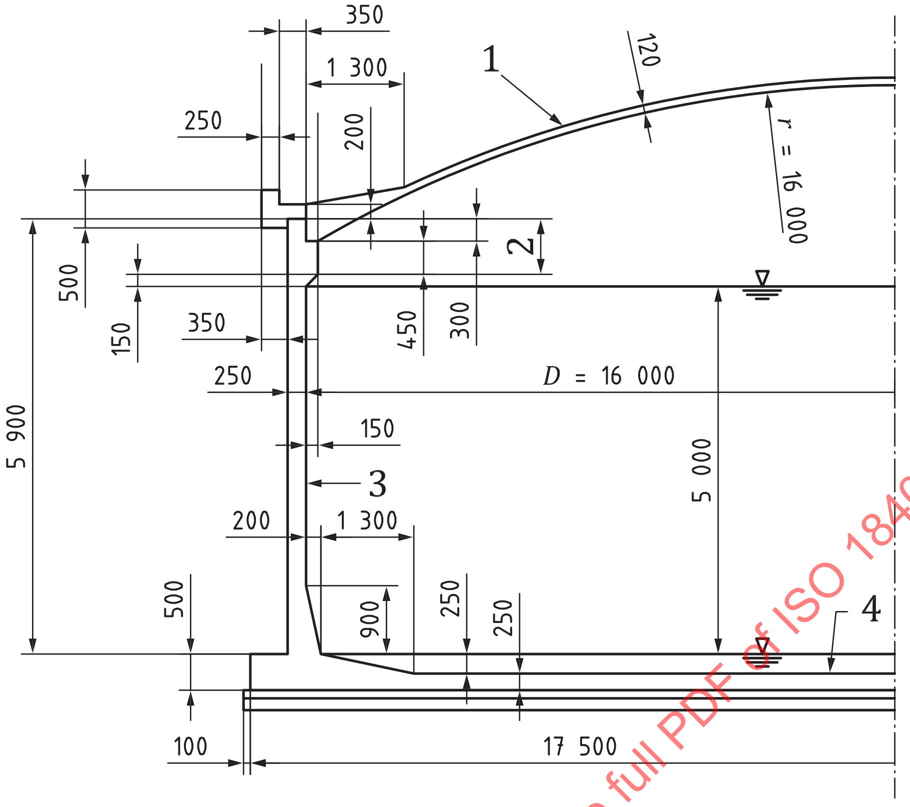
Effective water depth, H_e = 5 m

Wall thickness, t = 0,25 m

Nominal capacity, V_n = 1 000 m^3

Total capacity, V = 1 056 m^3

The detailed general shape is shown in Figure E.2 and arrangements of steel reinforcement and prestressing steel are given in Figures E.3 and E.4, respectively.



Key

1 dome roof (f'_{ck} = 24 MPa)

2 dome ring (f'_{ck} = 36 MPa)

3 wall (f'_{ck} = 36 MPa)

4 base slab (f'_{ck} = 30 MPa)

Figure E.2 — General shape in detail

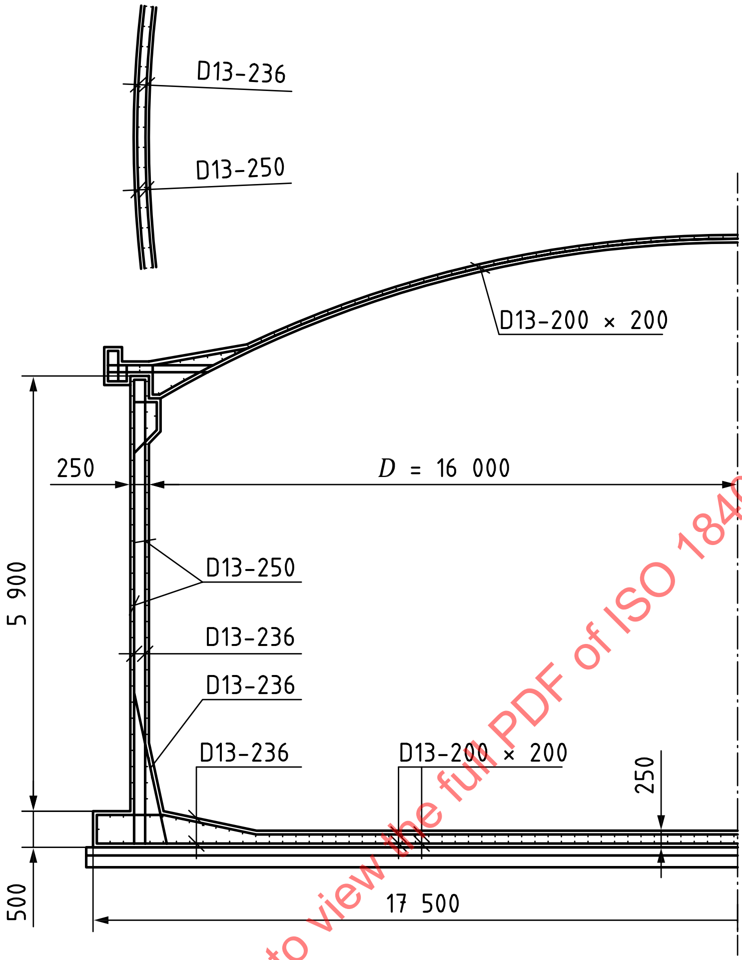


Figure E.3 — Arrangement of steel reinforcement

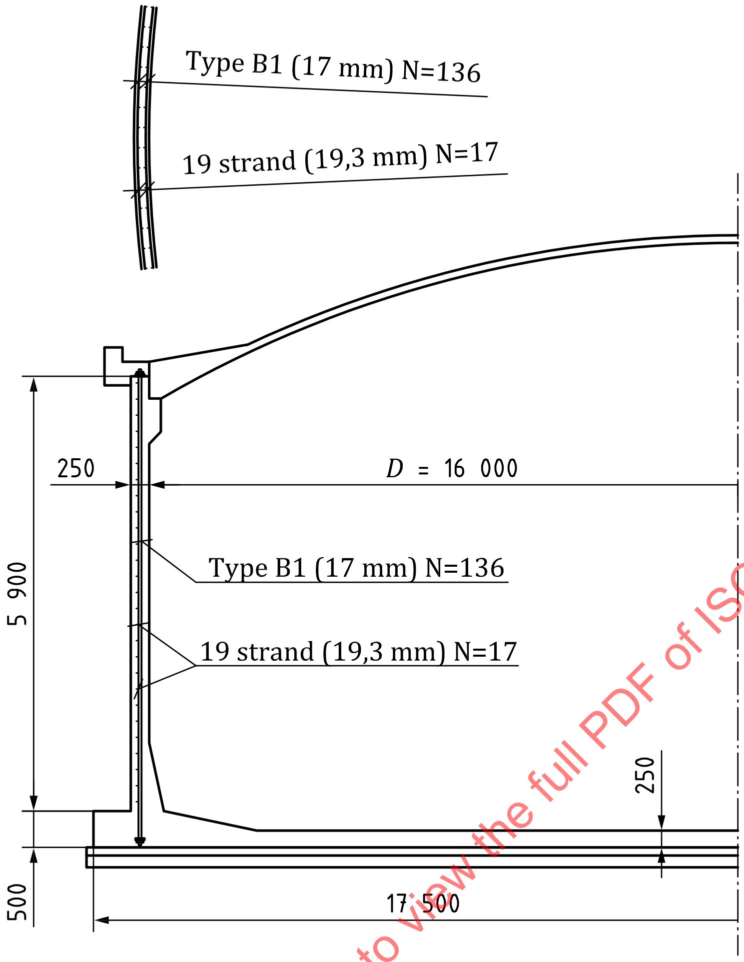


Figure E.4 — Arrangement of prestressing steel

E.2.2 Construction site and ground conditions

The construction site is located in Area A and the ground is assumed to be Type II as defined in Annex B.

E.2.3 Design loads

E.2.3.1 Classifications of design loads

The classifications of loads considered in the design calculation are listed in Table E.2.

Table E.2 — Classifications of loads

Classification	Load
Primary load	Deadweight
	Imposed load
	Hydrostatic pressure
	Prestressing force
Subsidiary load	Effect of earthquake
	Effect of temperature
Particular load	Earth pressure

E.2.3.2 Load type and load intensity

a) Deadweight

The unit weights for estimating the deadweight are given in Table E.3.

Table E.3 — Unit weight

Dimensions in kN/m^3

Type	Unit weight
Prestressed concrete	24,5
Reinforced concrete	24,5
Water	10,0

b) Imposed loads

The imposed loads are assumed as follows (in which a load of 0,5 kN/m^2 is considered, assuming maintenance staff is on the roof for inspection, etc.) after completion.

Imposed load on the dome 0,5 kN/m^2

Snow load on the dome 0,0 kN/m^2

Other loads on the dome 0,0 kN/m^2

c) Hydrostatic pressure

The hydrostatic pressure, p_w, is calculated by Formula (E.1).

$$p_w = 10\ kN/m^3 \times 5\ m = 50\ kN/m^2 \quad (E.1)$$

E.2.4 Materials properties

E.2.4.1 Concrete

Properties of concrete used in the PC water tank are listed in Table E.4.

Table E.4 — Concrete

	Base slab	Wall	Dome
Design strength, f'_{ck} (MPa)	30	36	24
Elastic modulus, E_c (GPa)	28	29,8	25
Poisson's ratio, ν	0,2	0,2	0,2
Linear expansion coefficient, α_e (1/°C)	$1{,}0 \times 10^{-5}$	$1{,}0 \times 10^{-5}$	$1{,}0 \times 10^{-5}$

E.2.4.2 Prestressing steel

a) Prestressing strand

Prestressing strand: 19 strand (ϕ_p = 19,3 mm)

Nominal cross-sectional area: 2,437 cm^2

Elasticity modulus, E_p: 200 GPa

b) Prestressing bar

Prestressing bar: Type B No.1 (ϕ_p = 17 mm)

Nominal cross-sectional area: 2,270 cm^2

Elasticity modulus, E_p: 200 GPa

E.2.4.3 Reinforcing bars

Ribbed bar: SD345 (f_{sy} = 345 MPa)

Elasticity modulus, E_s: 200 GPa

E.2.4.4 Stress loss of prestressing steel

The values for estimating stress loss of prestressing steel are listed in Table E.5.

Table E.5 — Values related to stress loss of prestressing steel

		Prestressing bar	Prestressing strand
Friction coefficient	Per unit angular change, μ (1/rad)	0,30	0,30
	Per unit length, λ (1/m)	0,003	0,004
Relaxation ratio, γ (%)		3	5
Drying shrinkage, ε_s		18×10^{-5}	
Creep factor, φ	Dome ring	2,8 (During prestressing, average: 4–7 days)	
	Wall	2,5 (During prestressing, average 14 days)	

E.2.5 Stress limit

Stress limits of materials are given in Tables E.6 to E.9 and their augmentations are listed in Tables E.10 and E.11.

Table E.6 — Stress limit of concrete for prestressed concrete members

Dimensions in MPa

Design compressive strength of concrete		36
Flexural compressive stress	Immediately after prestressing	17,4
	Other	13,8
Axial compressive stress	Immediately after prestressing	13,1
	Other	10,0
Flexural tensile stress	Immediately after prestressing	1,38
	Tank full	0,0
	Tank empty	0,72
Diagonal tensile stress		0,86

Table E.7 — Stress limit of concrete for reinforced concrete members

Dimensions in MPa

Design strength	24	30
Flexural compressive stress limit	9,0	11,0
Axial compressive stress limit	6,5	8,5

Table E.8 — Stress (load) limit of prestressing bar

Dimensions in kN

Type	17 mm Type B, No. 1
Under design load	147
Immediately after prestressing	172
During prestressing	190

Table E.9 — Stress limit of steel reinforcement

Dimensions in MPa

Type			SD345
Yield strength			345
Tensile stress limit	Standard member	Immediately after prestress-ing	245
		Other	196

Table E.10 — Augmentation of tensile stress limits of concrete

Dimensions in MPa

Design strength	24	30	36
Primary load + particular load equivalent to primary load + effect of temperature	1,52	1,70	1,88
Primary load + particular load equivalent to primary load + effect of an earthquake	2,02	2,20	2,38

Table E.11 — Augmentation (extra coefficient) of stress limits

Load combination	Extra coefficient
Primary load + particular load equivalent to primary load + effect of temperature (temperature difference)	1,15
Primary load + particular load equivalent to primary load + effect of an earthquake	1,50

E.2.6 Material strength

Design strengths of concrete, design loads of prestressing steel and design strength of steel reinforcement are given in Tables E.12 to E.15, respectively.

Table E.12 — Design strengths of concrete for reinforced concrete members

Dimensions in MPa

Concrete type	24	30
Design strength (f'_{ck})	24	30

Table E.13 — Design strengths (loads) of prestressing bars

Dimensions in kN

In tension	Tensile load (P_u)	245
	Yield load (P_y)	211

Table E.14 — Design strengths (loads) of prestressing strands

Dimensions in kN

In tension	Tensile load (P_u)	451
	Yield load (P_y)	387

Table E.15 — Design strength of steel reinforcement, MPa

Dimensions in MPa

In tension	Yield strength (f_{sy})	345

E.2.7 Material factor

Material factors for concrete and steel are given in Table E.16.

Table E.16 — Material factor

	γ_m
Concrete	1,3
Steel	1,0

E.2.8 Minimum reinforcement and minimum cover depth

The minimum reinforcement ratios and the minimum concrete cover to embedded steel reinforcement are given in Tables E.17 and E.18, respectively.

Table E.17 — Minimum reinforcement ratio

Member		Minimum reinforcement ratio, %
	Dome roof and dome ring	0,250
Wall	Circumferential direction of haunch at the wall bottom	0,550
	Other	0,250
Base slab	Centre	0,426
	Periphery	0,368

NOTE The minimum reinforcement ratio of the base slab is determined by the following formulae:

Centre (t_s = 0,25 m): 0,45 − (t_s − 0,15) 0,2/0,85 = 0,426 %

Periphery (t_s = 0,5 m): 0,45 − (t_s − 0,15) 0,2/0,85 = 0,368 %

Table E.18 — Minimum cover depth for steel reinforcement

Dimensions in mm

Member		Minimum cover depth
Dome roof and dome ring		30
Wall	Outside	30
	Inside	30
Base slab	Top side	30
	Under side	40

E.3 Design of the dome roof

E.3.1 Design conditions

The structural details of the dome roof at its wall joint is depicted in Figure E.5. The load combination for design calculation is given in Table E.19.

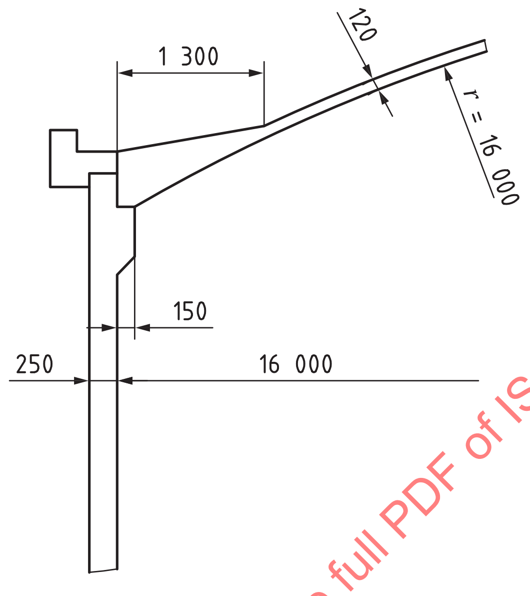


Figure E.5 — Details of the dome roof

Table E.19 — Load combination

Load	Deadweight	Imposed load
Ordinary conditions	○	○

E.3.2 Cross-section force acting on the dome

E.3.2.1 Calculation of the dome load

Calculation of dome loads is summarized in Figure E.6.

Half angle of the dome, α_d = 30,0°

Dome radius, $r = \dfrac{S_d}{2} \times \dfrac{1}{\sin \alpha_d} = 16{,}0\ \text{m}$

Dome rise, $h_d = r\,(1 - \cos\alpha_d)$ = 2,144 m

Dome surface area, $A_d = 2 \times \pi \times r \times h_d$ = 215,498 m^2

(Average thickness of the middle part of the shell = 0,12 m)

Weight per unit area, q_d = 0,12 × 24,5 = 2,94 kN/m^2

Imposed load, q_l = 0,5 kN/m^2

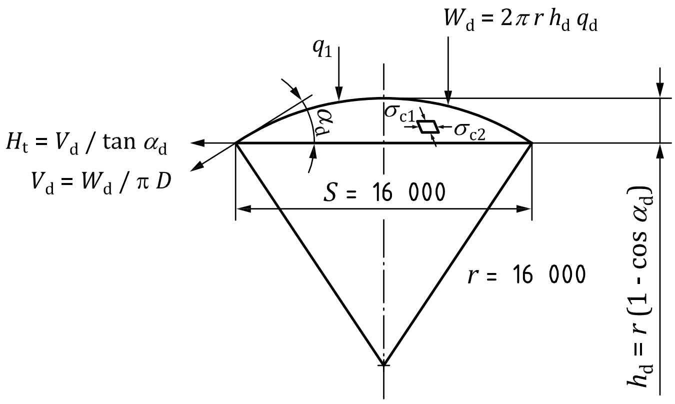


Figure E.6 — Calculation of dome loads

E.3.2.2 Stress generated in the dome

a) Membrane stress due to deadweight and imposed load [see Formulae (E.2) to (E.5)]:

$$\sigma_{c1} = q_d\, r \times \frac{1}{1+\cos\phi_d} \times \frac{1}{A_1} \tag{E.2}$$

$$\sigma_{c2} = q_d\, r \times \frac{\cos^2\phi_d + \cos\phi_d - 1}{1+\cos\phi_d} \times \frac{1}{A_2} \tag{E.3}$$

$$\sigma_{a1} = q_1\, r \frac{1}{2} \times \frac{1}{A_1} \tag{E.4}$$

$$\sigma_{a2} = q_1\, r \times \frac{\cos 2\phi_d}{2} \times \frac{1}{A_2} \tag{E.5}$$

where

σ_{c1}, σ_{a1} are the meridian stresses of the dome;

σ_{c2}, σ_{a2} are the parallel stresses of the dome;

A_1 is the cross-sectional area of the dome in the meridian direction (12 cm × 100 cm = 1 200 cm^2);

A_2 is the cross-sectional area of the dome in the parallel direction (12 cm × 100 cm = 1 200 cm^2);

ϕ_d is the angle of an arbitrary point of the dome from the rotation axis;

r is the dome radius;

q_d is the deadweight of the dome (= 2,94 kN/m^2);

q_l is the imposed load acting on the dome (= 0,5 kN/m^2).

b) Resultant stress

Resultant stresses are summarized in Table E.20.

Table E.20 — Resultant stress

ϕ_d deg	σ_1 MPa	σ_2 MPa
0,0	0,23	0,23
5,0	0,23	0,23
10,0	0,23	0,22
15,0	0,23	0,21
20,0	0,24	0,19
25,0	0,24	0,17
30,0	0,24	0,15

σ_{max} = 0,24 MPa < 6,5 MPa O.K.

σ_{min} = 0,15 MPa < 6,5 MPa O.K.

E.3.3 Steel reinforcement arrangement of the dome

Steel reinforcement for the dome, which is determined based on the minimum reinforcement ratio, is given in Table E.21.

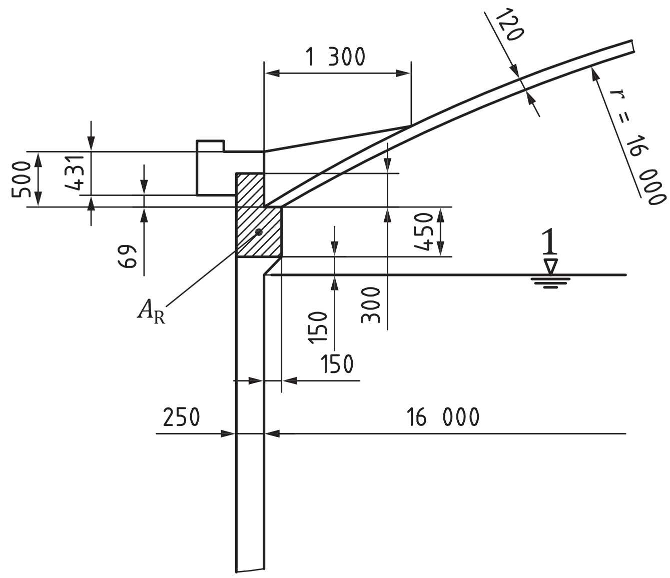
Table E.21 — Arrangement of steel reinforcement

	Steel reinforcement arrangement
Centre	D6@100 × 100 A_s = 0,317 × 1 000/100 = 3,17 cm^2 p = 3,17/(100 × 12) = 0,264 % > 0,25 % OK
Periphery	D13@200 × 200 double arrangement A_s = 2 × 1,267 × 1 000/200 = 12,67 cm^2 p = 12,67/(100 × 50) = 0,253 % > 0,25 % OK

E.4 Design of the dome ring

E.4.1 Design conditions

Design conditions of the dome ring are shown in Figure E.7. The load combination for the dome design is presented in Table E.22.



Key

1 H.W.L.

Figure E.7 — Geometry

Table E.22 — Load combination

Load	Deadweight of the dome	Prestressing force	Imposed load
Ordinary condition	○	○	○

E.4.2 Calculation of horizontal thrust

E.4.2.1 Loads on the dome

Loads acting on the dome are calculated as follows (see Formula (E.6)):

$$W = W_{d1} + W_{d2} + W_l \tag{E.6}$$

where

W_{d1} is the deadweight of the spherical shell (average thickness = 0,12 m):

$= q_d A_d$;

$= q_d 2\pi r^2(1 - \cos 30°)$;

$= 0{,}12 \times 24{,}5 \times 2\pi \times 16{,}0^2 \times (1 - \cos 30°)$;

$= 633{,}6$ kN;

W_{d2} is the deadweight of the widened dome periphery:

$= 2\pi r' A \rho_c$;

$= 2 \times \pi \times 7{,}567 \times 0{,}280 \times 24{,}5$;

$= 326{,}1$ kN;

W_l is the imposed load on the dome:

$= q_l\, A_d$;

$= q_l \times \pi/4 \times S_d^2$;

$= 0{,}5 \times \pi/4 \times 16{,}0^2$;

$= 100{,}5$ kN;

W $= W_{d1} + W_{d2} + W_l$;

$= 633{,}6 + 326{,}1 + 100{,}5$;

$= 1\,060{,}2$ kN.

E.4.2.2 Dome horizontal thrust

The dome horizontal thrust is calculated using Formula (E.7):

$$H_t = \frac{W}{\pi S_d \tan 30°} \quad \text{(E.7)}$$

where

H_t is the horizontal thrust of the dome;

W is the dome load;

S_d is the dome span (equal to the tank diameter).

Thus,

$$H_t = \frac{1\,060{,}2}{\pi \times 16{,}0 \times \tan 30°}$$

$$= 36{,}532 \text{ kN/m}$$

E.4.3 Design of prestressing force

E.4.3.1 Centre positions of prestressing steel

Inside diameter of the tank	16,0 m
Tank wall thickness	0,25 m
Outside diameter of a vertical sheath	0,029 m
Twice the thickness of a horizontal sheath	0,003 m
Diameter of prestressing steel	0,019 3 m

$D_p = 16{,}3013$ m

$R_p = 8{,}150\,7$ m

$R_o = 8{,}25$ m

$R_i = 8{,}0$ m

E.4.3.2 Section size of pilaster

Section size of the pilaster is shown in Figure E.8.

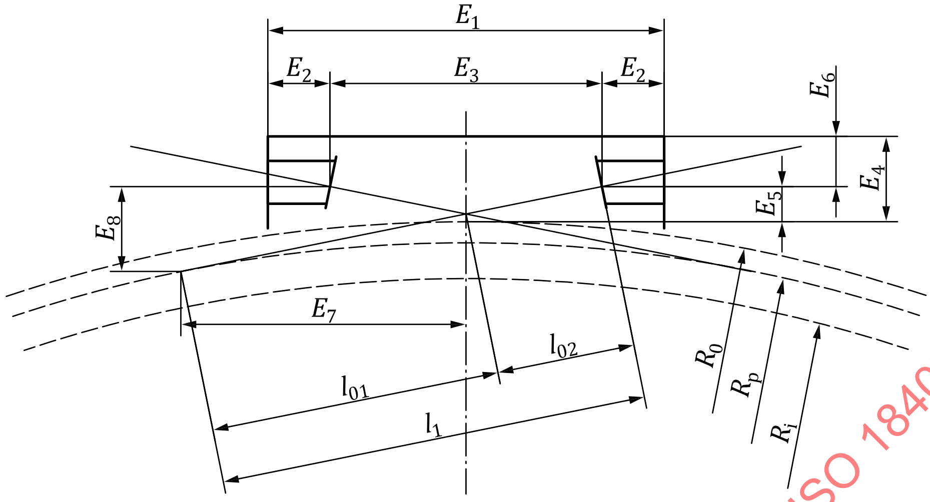


Figure E.8 — Section size of pilaster

E_1 = 2,0 m	E_2 = 0,2 m
E_3 = 1,6 m	E_4 = 0,289 m
E_5 = 0,12 m	E_6 = 0,169 m
E_7 = 1,241 m	E_8 = 0,314 m
l_1 = 2,065 m	
l_{01} = 1,255 m	l_{02} = 0,809 m
Concrete cover = 85,7 mm	ϕ = 8,757°

E.4.3.3 Required prestressing force

The prestressing force to be applied to the dome ring should be the sum of the horizontal dome thrust and an extra compressive force of 1 MPa [see Formula (E.8)].

$$F_d = F_1 + F_2 \tag{E.8}$$

where

F_d is the required prestressing force;

F_1 is the ring tensile force due to horizontal thrust;

F_2 is the ring tensile force due to extra compressive force.

Thus,

$F_1 = H_t S_d/2$

$= 36{,}532 \times 16{,}0/2$

$= 292{,}3$ kN

where

H is the horizontal dome thrust;

S_d is the dome span (equal to the tank diameter).

$F_2 = 1\,000 \times A_R$

$= 1\,000 \times (0{,}25 \times 0{,}75 + 0{,}15 \times 0{,}45)$

$= 255{,}0$ kN

where A_R is the cross-sectional area of the dome ring.

$F_d = 292{,}3 + 255{,}0$

$= 547{,}3$ kN

E.4.3.4 Calculation of the effective tensile force of prestressing steel

a) Prestressing force and stress of prestressing steel

Tensile force and stress of prestressing steel (single strand) are summarized in Table E.23.

Table E.23 — Single strand (for grouting)

Prestressing steel	Cross-sectional area, A_p mm^2	Prestressing force, P_i kN	Stress MPa
19-wire 19,3 mm	243,7	310	1 272

b) Friction loss of prestressing steel

One prestressing strand is used for a centre angle of 180° and tensioned at two of the four anchoring columns. The tensioning positions of adjacent prestressing strands are staggered.

Since each strand is tensioned simultaneously at both ends, the friction angle is calculated within the range of 180°/2; that is, 90°.

$\alpha = 90{,}0 - 8{,}757 = 81{,}243°$

$81{,}243° \times \pi/180 = 1{,}418$ rad

$\mu = 0{,}30$ (friction coefficient per radian of angular change, see Table 3)

$\lambda = 0{,}004$ (friction coefficient per metre of steel length, see Table 3)

$l_1 = 2{,}065$ m (length of the linear portion of steel)

$l_2 = R_p \times \alpha = 8{,}151 \times 1{,}418$

$= 11{,}557$ m (length of the arc portion of steel)

1/2 of the length of a prestressing strand

$l = l_1 + l_2 = 13{,}622$ m

$f_i = f_1 \times e^{(\lambda l_1)} = f_1 \times 1{,}008\,3\, f_1 =$

$f_2 \times e^{(\mu \cdot \alpha + \lambda l_2)} = f_2 \times 1{,}602\,6\, f_i =$

$f_2 \times 1{,}6159$

$\therefore (f_2 = f_i \times 0{,}618\ 9)$

c) Reduction in the tensile stress of prestressing steel due to anchor set loss

Anchor set loss is calculated as follows:

Prestressing steel: 19-wire strands, $\phi_p = 19{,}3$ mm

Set length: $\Delta l = 3{,}5$ mm

1) Ends

$P_i = 310$ kN

2) Prestressing force at the end of the linear portion

$P_2 = P_i \times 0{,}991\ 8 = 307{,}5$ kN

3) Prestressing force at the centre point

$P_4 = P_2 \times 0{,}624\ 0 = 191{,}8$ kN

4) Prestressing force at the limit point of the influence of set loss

$E_p = 200$ GPa

$A_{EP} = \Delta l \times A_p \times E_p = 170\ 590$ kN·mm

Reduced load, $\Delta P = 25{,}0$ kN

$P_3 = 307{,}5 - 25{,}0 = 282{,}5$ kN

$\leq P_{at} = 316{,}0$ kN OK

5) Prestressing force at the end of the linear portion after set loss

Reduced load, $\Delta P = 25{,}0$ kN

$P'_2 = 282{,}5 - 25{,}0 = 257{,}5$ kN

6) End prestressing force after set loss

Reduced load, $\Delta P = 1{,}6$ kN

$P'_1 = 257{,}5 - 1{,}6 = 256{,}0$ kN

7) Average prestressing force

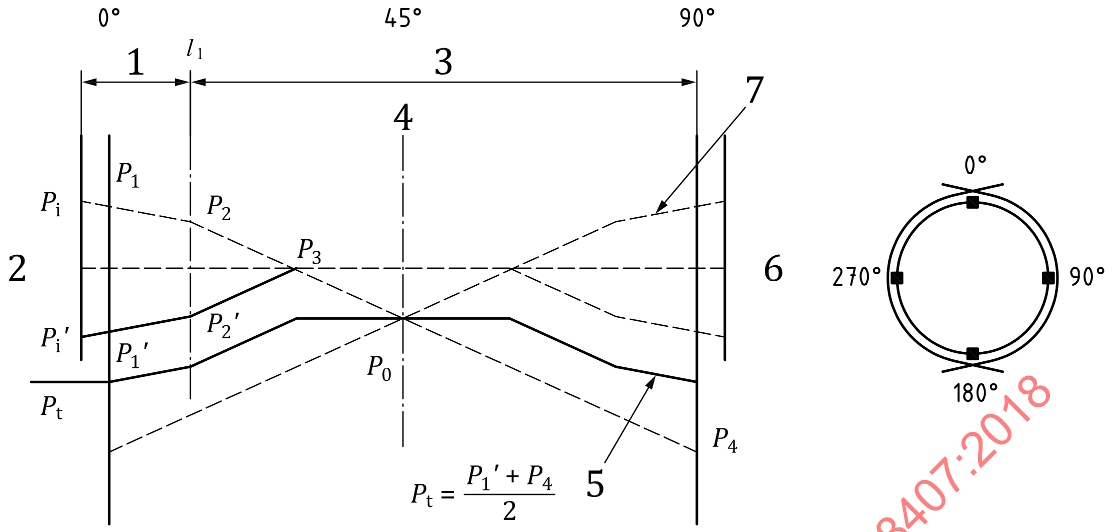
$P_t = (P'_1 + P_4)/2 = 223{,}9$ kN

Average stress

$\sigma'_{pt} = P_t/A_p = 918{,}8$ MPa

d) Distribution of prestressing forces with friction losses

Distribution of prestressing forces with friction losses is given in Figure E.9.



Key

1 linear
2 anchorage
3 arc
4 centre
5 average
6 farthest portion
7 tensioning force distribution by the adjacent strand

Figure E.9 — Distribution of prestressing forces with friction losses

e) Reduction in tensile stress of the prestressing steel due to elastic deformation of concrete

Reduction of the tensile stress in the prestressing steel is calculated and the results are summarized in Table E.24.

$$\Delta\sigma_p = \frac{1}{2} n \sigma'_{cpg} \quad \text{(E.9)}$$

where

$\Delta\sigma_p$ is the reduction in tensile stress of the prestressing steel due to elastic deformation of concrete;

n is the elastic modulus ratio (= E_p/E_c);

E_p is the elastic modulus of prestressing steel;

E_c is the elastic modulus of concrete at the prestressing age;

σ'_{cpg} is the concrete stress at the centroid of prestressing steel immediately after prestressing.

$$\sigma'_{cpg} = \frac{F_d}{A_R} \times \frac{1}{\eta} \quad \text{(E.10)}$$

where

F_d is the effective prestressing force of ring beam (= 547,270 kN);

A_R is the cross-sectional area of ring beam (= 255 000 mm^2);

η is the virtual effectiveness factor (= 0,85).

The stress of prestressing steel immediately after prestressing is:

$$\sigma_{pt} = \frac{P_t}{A_p} - \Delta\sigma_p \tag{E.11}$$

Table E.24 — Calculated results for the single strand (for grouting)

Prestressing steel	σ'_{cpg} MPa	$\Delta\sigma_p$	σ_{pt}
		MPa	
19-wire strand, 19,3 mm	2,5	8,5	910,3

f) Tensile stress loss of prestressing steel due to drying shrinkage, creep and relaxation

1) Tensile stress loss due to creep and drying shrinkage is calculated by Formula (6).

2) Tensile stress loss due to relaxation is calculated by Formula (7).

The results are given in Table E.25.

Table E.25 — Tensile stress loss of prestressing steel due to drying shrinkage, creep and relaxation

Prestressing steel	Elastic modulus ratio	Stress of permanent load	Loss due to drying shrinkage and creep	Loss due to steel relaxation
	$n = E_p/E_c$	σ'_{cp}	$\Delta\sigma_{p\phi}$	$\Delta\sigma_{p\gamma}$
		MPa	MPa	
19-wire strand, 19,3 mm	6,711	−1,0	61,2	45,5

g) Effective tensile force under the design loads

Calculation of effective stress

$\sigma_{pe} = \sigma_{pt} - (\Delta\sigma_{p\phi} + \Delta\sigma_{p\gamma})$

Calculation of effective tensile force

$P_e = \sigma_{pe} \times A_p$

Calculation of effectiveness factor

$\eta = \sigma_{pe}/\sigma_{pt}$

Available ratio (C) of effective prestressing force due to end prestressing force

$C = \sigma_{pi}/\sigma_{pe}$

The calculated results are given in Table E.26.

Table E.26 — Effective tensile force under the design loads

Prestressing steel	Effective tensile stress, σ_{pe}	Effective tensile force, P_e	Effectiveness factor, η	Available ratio C
	MPa	kN		
19-wire strand, 19,3 mm	803,6	195,8	0,883	1,583

E.4.3.5 Prestressing steel arrangement in the circumferential direction

Required prestressing force, F_d = 547,3 kN

Required number of strands, N = 547,3 × 1,583/310 = 2,8

As a result, three 19-wire strands (19,3 mm) will be placed.

E.5 Design of wall

E.5.1 Design conditions

Structural details of the wall are shown in Figure E.2. The load combinations summarized in Table 18 are applied.

E.5.2 Calculation of cross-section force

E.5.2.1 Basic formula and solution

Assuming an axisymmetric cylindrical shell, the cross-section force at the bottom edge of the wall is calculated based on Formulae (67) to (69). For the tank wall, the characteristic value, β, is 0,914 m^{-1}, the flexural rigidity, K, is 40 419 kN·m and the radius of the wall from the centre, R, is calculated as (16,0 + 0,25)/2 = 8,125 m.

When M_0 and Q_0 act on the bottom of a cylindrical shell (x = 0) with no restraint, the integration constants under the conditions indicated in Formula (71) are determined by Formula (72). The results are given as follows:

$$X_1(2\phi)=\frac{\cosh 2\phi-\cos 2\phi}{\cosh 2\phi+\cos 2\phi-2}=1{,}000\ 84$$

$$X_2(2\phi)=\frac{\sinh 2\phi+\sin 2\phi}{\cosh 2\phi+\cos 2\phi-2}=1{,}000\ 70$$

$$X_3(2\phi)=\frac{\sinh 2\phi-\sin 2\phi}{\cosh 2\phi+\cos 2\phi-2}=1{,}000\ 58$$

$$Y_1(2\phi)=\frac{1-\cos 2\phi}{\cosh 2\phi+\cos 2\phi-2}=0{,}000\ 42$$

$$Y_2(2\phi)=\frac{\cosh 2\phi-1}{\cosh 2\phi+\cos 2\phi-2}=1{,}000\ 42$$

$$2\phi = 2\beta H = 2 \times 0{,}914 \times 5{,}000 = 9{,}140$$

E.5.2.2 M_0 and Q_0 at the wall bottom

When fixed, the restraining moment, M_0, and restraining shearing force, Q_0, are calculated from Formula (E.12):

$$W_x\big|_{x=0}=0,\ \frac{dW_x}{dx}\Big|_{x=0}=0 \qquad \text{(E.12)}$$

a) Triangular load (prestress equivalent to water pressure)

$$M_{01}=-\frac{\rho H}{2\beta^2}\times\frac{X_1(2\phi)-X_3(2\phi)/(\beta H)}{2X_2(2\phi)X_3(2\phi)-\left[X_1(2\phi)\right]^2}$$

$$=-\frac{10\times5,0}{2\times0,914^2}\times\frac{1,000\ 84-1,000\ 58/(0,914\times5,0)}{2\times1,000\ 70\times1,000\ 58\times1,000\ 84^2}$$

$$=-23,4\ \mathrm{kN\cdot m}$$

$$Q_{01}=\frac{\rho H}{2\beta^2}\times\frac{2\beta X_2(2\phi)-X_1(2\phi)/H}{2X_2(2\phi)X_3(2\phi)-\left[X_1(2\phi)\right]^2}$$

$$=-\frac{10\times5,0}{2\times0,914^2}\times\frac{2\times0,914\times1,000\ 70-1,000\ 84/5,0}{2\times1,000\ 70\times1,000\ 58\times1,000\ 84^2}$$

$$=48,7\ \mathrm{kN}$$

b) Uniformly distributed load

$g_0 = F_2/R = 250,0/8,125 = 30,77$ kN

$$M_{02}=-\frac{g_0}{2\beta^2}\times\frac{X_1(2\phi)}{2X_2(2\phi)X_3(2\phi)-\left[X_1(2\phi)\right]^2}$$

$$=-\frac{30,8}{2\times0,914^2}\times\frac{1,000\ 84}{2\times1,000\ 70\times1,000\ 58-1,000\ 84^2}$$

$$=-18,4\ \mathrm{kN\cdot m}$$

$$Q_{02}=\frac{g_0}{\beta}\times\frac{X_2(2\phi)}{2X_2(2\phi)X_3(2\phi)-\left[X_1(2\phi)\right]^2}$$

$$=\frac{30,8}{0,914}\times\frac{1,000\ 70}{2\times1,000\ 70\times1,000\ 58-1,000\ 84^2}$$

$$=33,7\ \mathrm{kN}$$

c) M_0 and Q_0 at the wall bottom

$M_0 = M_{01} + M_{02} = -(23,4 + 18,4) = -41,8$ kN·m

$Q_0 = Q_{01} + Q_{02} = 48,7 + 33,7 = 82,4$ kN

E.5.2.3 Bending moment, shearing force and circumferential axial force at an arbitrary point

w_x, M_x, Q_x and N_ϕ at an arbitrary point are determined as follows:

$$w_x=\frac{\rho(H-x)}{4\beta^4K}+\frac{g_0}{4\beta^4K}+C_1F_4+C_2F_3+C_3F_2+C_4F_1$$

$$M_x = -2\beta^2K\,(C_1F_1 + C_2F_2 - C_3F_3 - C_4F_4) \tag{E.13}$$

$$Q_x = -2\beta^3K\,[C_1(F_2 - F_3) + C_2(F_1 - F_4) - C_3(F_1 + F_4) - C_4(F_2 + F_3)]$$

$$N_\varphi=-\frac{Et}{R}w_x$$

where

$F_1 = \cosh\beta x \cos\beta x$;

$F_2 = \sinh\beta x \cos\beta x$;

$F_3 = \cosh\beta x \sin\beta x$;

$F_4 = \sinh\beta x \sin\beta x$.

$$C_1 = -\frac{-41{,}8}{2\times 0{,}914^2 \times 40\ 419} = 0{,}000\ 6$$

$$C_2 = \frac{0{,}914\times(-41{,}8)\times 1{,}000\ 70 + 82{,}4\times 0{,}000\ 42}{2\times 0{,}914^2 \times 40\ 419} = -0{,}000\ 6$$

$$C_3 = \frac{0{,}914\times(-41{,}8)\times 1{,}000\ 70 + 82{,}4\times 1{,}000\ 42}{2\times 0{,}914^2 \times 40\ 419} = 0{,}000\ 7$$

$$C_4 = -\frac{0{,}914\times(-41{,}8)\times 1{,}000\ 84 + 82{,}4\times 1{,}000\ 58}{2\times 0{,}914^2 \times 40\ 419} = -0{,}000\ 7$$

E.5.2.4 Stress due to deadweight

Stresses calculated at the wall bottom (x = 0) and at the middle point (x = 1,641 m) are summarized in Tables E.27 and E.28, respectively.

Table E.27 — Wall bottom (x = 0m)

	Calculation formula	Weight, W (kN/m)
Dome	(633,563 + 326,143)/(π × 16,25)	18,799
Dome rest	24,5 × (0,45 + 0,15/2) × 0,15	1,929
Corridor	24,5 × 0,25 × 0,2	1,225
ditto	24,5 × 0,35 × 0,3	2,573
ditto	24,5 × 0,25 × 0,2	1,225
Wall	24,5 × 0,25 × (0,3 + 0,6 + 5)	36,138
Haunch	24,5 × 0,2 × 0,9/2	2,205
ΣW		64,093

$\sigma_W = W/A$ = 64,093 × 1 000/450 000 = 0,14 MPa

Table E.28 — Middle point (x = 1,641 m)

	Calculation formula	Weight, W (kN/m)
Dome	(633,563 + 326,143)/(π × 16,25)	18,799
Dome rest	24,5 × (0,45 + 0,15/2) × 0,15	1,929
Corridor	24,5 × 0,25 × 0,2	1,225
ditto	24,5 × 0,35 × 0,3	2,573
ditto	24,5 × 0,25 × 0,2	1,225
Wall	24,5 × 0,25 × (0,3 + 0,6 + 5 − 1,641)	26,086
ΣW		51,837

$\sigma_w = W/A$ = 51,837 × 1 000/250 000 = 0,21 MPa

E.5.2.5 Stress due to imposed load

Stress due to the imposed load is calculated.

$w_l = q_l A_d$ = 1,97 kN/m

Wall bottom: $\sigma_l = w_l/A$ = 1,97 × 1 000/450 000 = 0,004 MPa

Intermediate portion: $\sigma_l = w_l/A$ = 1,97 × 1 000/250 000 = 0,008 MPa

E.5.2.6 Cross-section force incorporating the effects of the haunch at wall bottom and elastic fixation

Cross-section force incorporating the effect of elastic fixation is calculated and the results are summarized in Table E.29.

$$M_{0f} = (k_\alpha k_\beta) M_{0h} \tag{E.14}$$

where

M_{0f}	is the vertical bending moment at the wall bottom incorporating the effect of elastic fixation;
M_{0h}	is the vertical bending moment at the wall bottom incorporating the increased wall thickness at the wall bottom;
k_α	is the coefficient incorporating the characteristics of foundations;
k_β	is the coefficient incorporating the rigidity of base slab.

$k_\alpha k_\beta = 0{,}75$

$M_{0f} = 0{,}75 \times M_{0h}$

$= 0{,}75 \times 1{,}76\ M_{0c}$

$= 1{,}32\ M_{0c}$

Table E.29 — Cross-section force incorporating the effect of wall bottom haunch and elastic fixation

Load		Vertical bending moment at wall bottom when assuming a constant wall thickness of t M_{0c} (kN·m)	Correction factor	Vertical bending moment at wall bottom incorporating the effect of haunch and elastic fixation M_{0f} (kN·m)
Hydrostatic pressure		23,4	1,32	30,9
Circumferential pressure + earth pressure	Immediately after prestressing	−52,1	1,32	−68,8
	Under design loads	−45,2	1,32	−59,7
During an earthquake (Level 1)	Inertia force + dynamic water pressure	6,7	1,32	8,8
During an earthquake (Level 2)	Inertia force + dynamic water pressure	12,0	1,32	15,9

NOTE 1 The values during an earthquake is in the direction of 180°.

NOTE 2 The value immediately after prestressing = the value under design load/effectiveness factor (= 0,869).

E.5.3 Design of circumferential prestressing force

E.5.3.1 Centre positions of prestressing steel

The basic dimensions are the same as those given in E.4.3.1.

E.5.3.2 Section size of pilaster

The section sizes of the pilaster are the same as those defined in Figure E.8.

E.5.3.3 Calculation of the effective tensile force of prestressing steel

a) Prestressing force and stress of prestressing steel

Tensile force and stress of prestressing steel (single strand) are summarized in Table E.30.

Table E.30 — Single strand (for grouting)

Prestressing steel	Cross-sectional area, A_p (mm^2)	Prestressing force, P_i (kN)	Stress, (MPa)
19-wire, 19,3 mm	243,7	310	1 272

b) Friction loss of prestressing steel

One prestressing strand is used for a centre angle of 180° and tensioned at two of the four anchoring columns. The tensioning positions of adjacent prestressing strands are staggered.

Since each strand is tensioned simultaneously at both ends, the friction angle should be calculated within the range of 180°/2; that is, 90°.

$\alpha = 90{,}000 - 8{,}757 = 81{,}243°$

$81{,}243 \times \pi/180 = 1{,}418$ rad

μ = 0,30 (friction coefficient per radian of angular change)

λ = 0,004 (friction coefficient per metre of steel length)

l_1 = 2,065 m (length of the linear portion of steel)

$l_2 = R_p \times \alpha = 8{,}151 \times 1{,}418$

= 11,557 m (length of the arc portion of steel)

1/2 of the length of a prestressing strand

$L = l_1 + l_2 = 13{,}622$ m

$f_i = f_1 \times e^{(\lambda\, l_1)} = f_1 \times 1{,}0083$

$f_1 = 2 \times e^{(\mu\, \alpha + \lambda\, l_2)} = f_2 \times 1{,}6026$

$f_i = f_2 \times 1{,}6159$

$\therefore (f_2 = f_i \times 0{,}6189)$

c) Reduction of tensile stress of prestressing steel due to anchor set loss is calculated as follows:

Prestressing steel: 19-wire strands, ϕ_p = 19,3 mm

Set length: Δl = 3,5 mm

1) Ends

P_i = 310 kN

2) Prestressing force at the end of the linear portion

$P_2 = P_i \times 0{,}991\,8 = 307{,}5$ kN

3) Prestressing force at the centre point

$P_4 = P_2 \times 0{,}624\,0 = 191{,}8$ kN

4) Prestressing force at the limit point of the influence of set loss

E_p = 200 GPa

$A_{ep} = \Delta l \times A_p \times E_p$ = 170 590 kN·mm

Reduced load, ΔP = 25,0 kN

P_3 = 307,5 − 25,0 = 282,5 kN

$\leq P_{at}$ = 316,0 kN OK

5) Prestressing force at the end of the linear portion after set loss

Reduced load, ΔP = 25,0 kN

P'_2 = 282,5 − 25,0 = 257, 5kN

6) End prestressing force after set loss

Reduced load, ΔP = 1,6 kN

P'_1 = 257,5 − 1,6 = 256,0 kN7) Average prestressing force

$P_t = (P'_1 + P_4)/2$ = 223,9 kN

Average stress

$\sigma'_{pt} = P_t / A_p$ = 918,8 MPa

d) Distribution of prestressing forces with friction loss

Distribution of prestressing forces with friction losses is given in Figure E.9.

e) Reduction in tensile stress of the prestressing steel due to elastic deformation of concrete

$$\Delta\sigma_p = \frac{1}{2} n \sigma'_{cpg} \tag{E.15}$$

where

$\Delta\sigma_p$ is the reduction in tensile stress of the prestressing steel due to elastic deformation of concrete;

n is the elastic modulus ratio (= E_p/E_c);

E_p is the elastic modulus of prestressing steel;

E_c is the elastic modulus of concrete at the prestressing age;

σ'_{cpg} is the concrete stress at the centroid of prestressing steel immediately after prestressing.

σ'_{cpg} is calculated at the position where the prestressing force coincides with the average value; that is 1/3 of the water depth from the wall bottom.

$$\sigma'_{cpg} = \left[\frac{2}{3} \frac{H \rho R}{t} \times \sigma_{ext} \right] \frac{1}{\eta} \tag{E.16}$$

where

H is the design water depth;

ρ is the unit weight of water (= 10 kN/m^3);

R is the wall radius;

t is the wall thickness;

σ_{ext} is the extra compressive stress;

η is the virtual effectiveness factor (= 0,85).

The stress of prestressing steel immediately after prestressing is:

$$\sigma_{pt} = \frac{P_t}{A_p} - \Delta\sigma_p \qquad \text{(E.17)}$$

The calculated results are given in Table E.31.

Table E.31 — Reduction in the tensile stress of prestressing steel due to elastic deformation of concrete

Prestressing steel	σ'_{cpg} MPa	$\Delta_{\sigma p}$	σ_{pt}
		MPa	
19-wire strand, 19,3mm	2,5	8,2	910,6

f) Tensile stress loss of prestressing steel due to drying shrinkage, creep and relaxation

1) Tensile stress loss due to creep and drying shrinkage is calculated by Formula (6).

2) Tensile stress loss due to relaxation is calculated by Formula (7).

Tensile stress losses calculated are summarized in Table E.32.

Table E.32 — Tensile stress loss of prestressing steel due to drying shrinkage, creep and relaxation

Prestressing steel	Elastic modulus ratio $n = E_p/E_c$	Stress of permanent load σ'_{cp} MPa	Loss due to drying shrinkage and creep $\Delta\sigma_{p\varphi}$	Loss due to steel relaxation $\Delta\sigma_{p\gamma}$
			MPa	
19-wire strand, 19,3 mm	6,711	0,0	74,1	45,5

g) Effective tensile force under the design loads

Calculation of effective stress

$\sigma_{pe} = \sigma_{pt} - (\Delta\sigma_{p\varphi} + \Delta\sigma_{p\gamma})$

Calculation of effective tensile force

$P_e = \sigma_{pe} \times A_p$

Calculation of effectiveness factor

$\eta = \sigma_{pe}/\sigma_{pt}$

Available ratio (C) of effective prestressing force due to end prestressing force

$C = \sigma_{pi} / \sigma_{pe}$

The calculated results are given in Table E.33.

Table E.33 — Effective tensile force under the design loads

Prestressing steel	Effective tensile stress, σ_{pe} MPa	Effective tensile force, P_e kN	Effectiveness factor, η	Available ratio, C
19-wire strand, 19,3 mm	790,9	192,8	0,869	1,608

E.5.3.4 Required prestressing force

Inside diameter D = 16,0 m

Water depth H = 5,0 m

Wall thickness t = 0,25 m

Maximum hydrostatic water pressure, p_w = 50 kN/m^2

Maximum ring tensile force due to water pressure (see Figure E.10).

$$F_1 = p_w \frac{(D+t)}{2}$$

$$= 50 \times \frac{16{,}0 + 0{,}25}{2}$$

$$= 406{,}3 \text{ kN/m}$$

$$\Sigma F_1 = F_1 \times H/2$$

$$= 1\,015{,}6 \text{ kN}$$

Effective prestressing force for an extra compressive force of 1 000 kN/m^2 (see Figure E.10).

$$F_2 = 1\,000 \times t \times 1{,}0$$

$$= 1\,000 \times 0{,}25 \times 1{,}0$$

$$= 250{,}0 \text{ kN/m}$$

$$\Sigma F_2 = F_2 \times (H + 0{,}15)$$

$$= 250 \times (5{,}0 + 0{,}15)$$

$$= 1\,287{,}5 \text{ kN}$$

$$F_3 = 1\,000 \times t' \times 1{,}0$$

$$= 1\,000 \times 0{,}2 \times 1{,}0$$

$$= 200{,}0 \text{ kN/m}$$

t' Haunch width

$$\Sigma F_3 = F_3 \times H_h/2$$

