CEN/TC 250

Date: 2023-08

prEN 1993‑4‑2:2024

Secretariat: BSI

Eurocode 3 — Design of steel structures — Part 4-2: Tanks

Eurocode 3 — Calcul des structures en acier — Partie 4-2: Reservoirs

Eurocode 3 — Bemessung und Konstruktion von Stahlbauten — Teil 4-2: Tankbauwerke

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European foreword

This document (prEN 1993‑4‑2:2024), has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the Secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will supersede EN 1993‑4‑2:2007 and its amendments and corrigenda.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognize the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National Annexes.

0 Introduction

**0.1 Introduction to the Eurocodes**

The Structural Eurocodes comprise the following standards generally consisting of a number of Parts:

— EN 1990, Eurocode — Basis of structural and geotechnical design

— EN 1991, Eurocode 1 — Actions on structures

— EN 1992, Eurocode 2 — Design of concrete structures

— EN 1993, Eurocode 3 — Design of steel structures

— EN 1994, Eurocode 4 — Design of composite steel and concrete structures

— EN 1995, Eurocode 5 — Design of timber structures

— EN 1996, Eurocode 6 — Design of masonry structures

— EN 1997, Eurocode 7— Geotechnical design

— EN 1998, Eurocode 8— Design of structures for earthquake resistance

— EN 1999, Eurocode 9 — Design of aluminium structures

— New parts are under development, e.g. Eurocode for design of structural glass

**0.2 Introduction to the** **EN** **1993** **series**

EN 1993 applies to the design of buildings and civil engineering works in steel. It complies with the principles and requirements for the safety and serviceability of structures, the basis of their design and verification that are given in EN 1990 – Basis of structural and geotechnical design.

EN 1993 is concerned only with requirements for resistance, serviceability, durability and fire resistance of steel structures. Other requirements, e.g. concerning thermal or sound insulation, are not covered.

EN 1993 is subdivided in various parts:

EN 1993‑1, *Design of Steel Structures — Part 1: General rules and rules for buildings;*

EN 1993‑2, *Design of Steel Structures — Part 2: Bridges;*

EN 1993‑3, *Design of Steel Structures — Part 3: Towers, masts and chimneys;*

EN 1993‑4, *Design of Steel Structures — Part 4: Silos and tanks;*

EN 1993‑5, *Design of Steel Structures — Part 5: Piling;*

EN 1993‑6, *Design of Steel Structures — Part 6: Crane supporting structures;*

EN 1993‑7, *Design of steel structures — Part 7: Sandwich panels.*

EN 1993‑1 in itself does not exist as a physical document, but comprises the following 14 separate parts, the basic part being EN 1993‑1‑1:

EN 1993‑1‑1, *Design of Steel Structures — Part 1-1: General rules and rules for buildings;*

EN 1993‑1‑2, *Design of Steel Structures — Part 1-2: Structural fire design;*

EN 1993‑1‑3, *Design of Steel Structures — Part 1-3: Cold-formed members and sheeting;*

NOTE Cold formed hollow sections supplied according to EN 10219 are covered in EN 1993‑1‑1.

EN 1993‑1‑4, *Design of Steel Structures — Part 1-4: Stainless steel structures;*

EN 1993‑1‑5, *Design of Steel Structures — Part 1-5: Plated structural elements;*

EN 1993‑1‑6, *Design of Steel Structures — Part 1-6: Strength and stability of shell structures;*

EN 1993‑1‑7, *Design of Steel Structures — Part 1-7: Plate assemblies with elements under transverse loads;*

EN 1993‑1‑8, *Design of Steel Structures — Part 1-8: Joints;*

EN 1993‑1‑9, *Design of Steel Structures — Part 1-9: Fatigue;*

EN 1993‑1‑10, *Design of Steel Structures — Part 1-10: Material toughness and through-thickness properties;*

EN 1993‑1‑11, *Design of Steel Structures — Part 1-11: Tension components;*

EN 1993‑1‑12, *Design of Steel Structures — Part 1-12: Additional rules for steel grades up to S960;*

EN 1993‑1‑13, *Design of Steel Structures — Part 1-13: Beams with large web openings;*

EN 1993‑1‑14, *Design of Steel Structures — Part 1-14: Design assisted by finite element analysis.*

All subsequent parts EN 1993‑1‑2 to EN 1993‑1‑14 treat general topics that are independent from the structural type such as structural fire design, cold-formed members and sheeting, stainless steels, plated structural elements, etc.

All subsequent parts numbered EN 1993‑2 to EN 1993‑7 treat topics relevant for a specific structural type such as steel bridges, towers, masts and chimneys, silos and tanks, piling, crane supporting structures, etc. EN 1993‑2 to EN 1993‑7 refer to the generic rules in EN 1993‑1 and supplement, modify or supersede them.

**0.3 Introduction to** **EN** **1993‑4‑2**

EN 1993‑4‑2 gives design requirements for the structural design of tanks for the storage of liquid and liquified gas products. It gives design rules that supplement the generic rules in the many parts of EN 1993‑1. This document is intended for clients, designers, contractors and relevant authorities.

**0.4 Verbal forms used in the Eurocodes**

The verb “shall” expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may” expresses a course of action permissible within the limits of the Eurocodes.

The verb “can” expresses possibility and capability; it is used for statements of fact and clarification of concepts.

**0.5 National Annex for** **EN** **1993‑4‑2**

National choice is allowed in this standard where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing EN 1993‑4‑2 can have a National Annex containing all national choices to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

When no national choice is given, the default choice given in this standard is to be used.

When no national choice is made and no default is given in this standard, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by appropriate parties.

National choice is allowed in EN 1993‑4‑2 through notes to the following clauses:

|  |  |  |  |
| --- | --- | --- | --- |
| 4.3.2(3) | 4.3.3(2) | 4.4.1(3) | 4.4.2(4) |
| 4.4.3(2) | 4.11(4) | 7.1.3(1) | 8.4.7(1) |
| 10.2(1) | 10.2(9) | 10.3(2) |   |

National choice is allowed in EN 1993‑4‑2 on the application of the following informative annexes:

None

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

# Scope

## Scope of EN 1993‑4‑2

(1) EN 1993‑4‑2 provides rules for structural design of vertical cylindrical, conical and pedestal above-ground steel tanks for the storage of liquid and liquified gas products.

(2) EN 1993‑4‑2 is applicable to the design for resistance of cylindrical walls and flat bottoms constructed using unstiffened plates. The design of conical and dome roofs as shell structures (unsupported) or as supported on a structural framework (supported) are also covered.

(3) EN 1993‑4‑2 is only applicable to the requirements for resistance and structural stability of steel tanks.

(4) EN 1993‑4‑2 only covers steel tank structures in Tank Groups 1, 2 and 3, as defined in this document.

NOTE Tank Group 4 is not defined in this standard (see 3.1.41).

(5) This document is applicable to tanks within the following dimensional limits (see EN 1991‑4):

Tank aspect ratio *hS*/*d* < 10

Tank total height *hS* < 70 m

Tank diameter *d* < 100 m

(6) This standard includes suitable rules for the design of tanks intended to store solids suspended in a liquid, where the appropriate global density of the mixture is used.

NOTE Tanks used for the separation of mineral particles of different density fall into this category.

(7) EN 1993‑4‑2 does not apply to the following:

a) tanks with gross capacity less than 5 m3 (5 000 l);

b) dished-end tanks that have a diameter less than 5 m;

c) tanks with characteristic internal pressures above the liquid surface greater than 50 kPa (500 mbar)[[1]](#footnote-1) (see pressure equipment directive);

d) design metal temperatures outside the ranges defined in Clause 5, with −50 °C being the lowest temperature for the application of this document;

e) tanks of rectangular and other non-circular planforms;

f) tanks exposed to fire;

g) floating roofs and floating covers;

h) ancillary structures such as stairways, platforms, nozzles, piping and access doors.

(8) This document does not cover

a) the special requirements for seismic design of tanks,

b) the design of a supporting structure,

c) the design of ancillary structures such as stairways, platforms, pipe racks and ladders,

d) the design of an aluminium roof structure on a steel tank,

e) reinforced concrete foundations for steel tanks,

f) the design of a conical hopper,

g) the design of a transition junction between the base of a cylindrical shell wall and a conical hopper,

h) the design of a supporting ring girder in an elevated tank.

## Assumptions

(1) Unless specifically stated, EN 1990, the EN 1991 series and the EN 1993‑1 series apply.

(2) The design methods given in this document apply if:

— the execution quality is as specified in EN 1090‑2, and

— the construction materials and products used are as specified in the relevant parts of the EN 1993 series, or in the relevant material and product specifications.

(3) This standard applies to axisymmetric structures, but includes the effects of unsymmetrical actions (e.g. wind), and unsymmetrically supported tanks (e.g. on discrete supports).

(4) EN 1993‑4‑2 is intended to be used in conjunction with EN 1990, with EN 1991‑4, with the other Parts of EN 1991, with EN 1993‑1‑6 and EN 1993‑4‑1, with the other Parts of EN 1993, with EN 1992 and with the other Parts of EN 1994 to EN 1999 relevant to the design of tanks. Matters that are already covered in those documents are not repeated.

(5) Numerical values for partial factors and other reliability parameters are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and quality management applies.

# 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.

EN 1090‑2, Execution of steel structures and aluminium structures — Part 2: Technical requirements for steel structures

EN 1990:2023,[[2]](#footnote-2) Eurocode — Basis of structural and geotechnical design

EN 1991 (all parts), Eurocode 1 — Actions on structures

EN 1993 (all parts), Eurocode 3 — Design of steel structures

# Terms, definitions, symbols, sign conventions and units

## Terms and definitions

For the purposes of this document, the terms and definitions from EN 1990, ISO 8930 and the following apply.

3.1.1

axial direction

*x*

vertical direction on the shell wall

Note 1 to entry: For a cylindrical wall, the axial and meridional directions are identical.

3.1.2

axisymmetric shell

shell structure whose geometry is defined by rotation of a meridional line about a central axis

3.1.3

base ring or annular plate

structural member that passes around the circumference of the structure beneath the cylindrical shell wall and is required to ensure that the assumed boundary conditions are achieved in practice

3.1.4

catch basin

external tank structure to contain liquid that could escape by leakage or accident from the primary tank

Note 1 to entry: This type of structure is usually used where the primary tank contains toxic or hazardous liquids. A catch basin also effectively reduces the requirement for an extensive area of liquid containment around the tank.

3.1.5

circumferential direction

*θ*

horizontal tangent to the shell wall at any point

Note 1 to entry: It varies around the tank, lies in the horizontal plane and is tangential to the shell wall.

3.1.6

continuously supported tank

tank in which all positions around the circumference are supported in an identical manner

Note 1 to entry: Minor departures from this condition (e.g. a small opening) need not affect the applicability of the definition.

3.1.7

course

section of the height of a cylindrical wall constructed from a single plate thickness or between ring stiffeners, usually made up of several strakes (see 3.1.39)

Note 1 to entry: The cylindrical shell wall of the tank is formed by making horizontal joints between a series of short cylindrical sections, termed strakes, each formed by making vertical joints between individual curved plates.

3.1.8

curb angle

light ring attached to the top of the cylindrical shell wall of a tank

3.1.9

discrete support

situation in which a tank is supported using a local bracket or column, giving a limited number of narrow supports around the tank circumference

3.1.10

eaves junction

joint between a cylindrical wall and a roof structure

Note 1 to entry: See also curb angle and primary ring.

3.1.11

externally stiffened wall

tank wall with stiffeners attached to the outside of the tank wall

3.1.12

fixed shell roof

roof structure that is attached to the top of the cylindrical wall

Note 1 to entry: This term includes both roofs supported on rafters or a structural frame and an unstiffened shell roof (sometimes referred to as unsupported).

3.1.13

floating roof

roof structure that floats on the surface of the stored liquid, sliding up and down within the tank as the liquid level varies

3.1.14

gas/vapour pressure

pressure in the space above the surface of the stored liquid

3.1.15

ground supported tank

tank where the shell structure is uniformly supported on a horizontal foundation supported directly by the ground

3.1.16

hopper

converging section towards the bottom of a tank

Note 1 to entry: It is used to channel liquids towards a gravity discharge outlet (usually when they contain suspended solids).

3.1.17

horizontally corrugated wall

shell wall constructed from corrugated steel sheets where the troughs pass around the circumference of the tank

3.1.18

internally stiffened wall

tank wall with stiffeners attached to the inside of the tank wall

3.1.19

isotropic wall

shell wall of a tank constructed from flat rolled steel sheet

3.1.20

joint efficiency factor

ratio of the membrane resistance of a welded or bolted joint to the yield membrane resistance of the parent plate

3.1.21

junction

point at which any two or more shell segments are connected

Note 1 to entry: It can include a stiffener or can have no stiffener: the point of attachment of a ring stiffener to the shell is treated as a junction.

3.1.22

maximum design liquid level

MDLL

highest liquid level in a tank which is used as an accidental load design situation

Note 1 to entry: This is higher than the maximum normal operating level (see Figure 3.1).

3.1.23

maximum normal operating liquid level

MNOL

highest liquid level in a tank under normal operating conditions and used in load combinations according to EN 1990:2023, Clause A.4

Note 1 to entry: This is lower than the maximum design liquid level (see Figure 3.1)

3.1.24

meridional direction

*ϕ*

tangent to the shell wall at any point in a plane that passes through the axis of the tank

Note 1 to entry: It varies according to the structural element being considered. For a cylindrical wall, the axial and meridional directions are identical, and axial is generally the preferred term.

3.1.25

middle surface

middle of the shell wall at any point, such that under elastic conditions, this surface is stress free when the shell is subject only to bending in any direction

3.2.26

minimum operating liquid level

min NOL

lowest liquid level in a tank under normal operations

Note 1 to entry: See Figure 3.1.

3.1.27

purlin

circumferentially oriented structural member supporting a roof and supported itself on rafters

3.1.28

primary ring

ring located near the top of the cylindrical wall or at the eaves junction

Note 1 to entry: It can also be referred to as the top ring or wind girder. Where there is a roof, the primary ring also serves the purpose of forming the junction between the roof and cylindrical wall.

3.1.29

rafter

radially oriented structural member supporting a roof structure

3.1.30

rib

local member that provides a primary load-carrying path for loads causing bending down the meridian of a shell, representing a generator of the shell of revolution

Note 1 to entry: It is used to distribute transverse loads on the structure by bending action.

3.1.31

ring girder or ring beam

circumferential stiffener which has bending stiffness and strength normal to the plane of the circular section of a shell as well as in that plane

Note 1 to entry: It is a primary load-carrying element, used to distribute local vertical loads into the shell.

3.1.32

ring stiffener

local stiffening member that passes around the circumference of the structure at a given point on the meridian

Note 1 to entry: It is assumed to have no stiffness in the meridional plane of the structure. It is provided to increase the stability or to introduce local loads, not as a primary load-carrying element.

3.1.33

RLG

refrigerated liquefied gas

3.1.34

secondary ring stiffener

ring stiffener on a tank wall located on the cylindrical wall below the primary ring

Note 1 to entry: There can be multiple secondary rings. The purpose of the secondary ring is to increase the buckling resistance of the cylindrical shell under external pressure and wind.

3.1.35

separation of ring stiffeners

centre to centre distance between the circumferential axes of two adjacent ring stiffeners

3.1.36

shell

vertical wall of a ground-supported cylindrical tank

Note 1 to entry: The term shell is often used in the tank industry to refer to the vertical wall of a ground-supported cylindrical tank. This usage is slightly confusing when compared with the general definition of a shell (see EN 1993‑1‑6) which has more general application. To avoid confusion, the term “shell” is used in this standard where appropriate to the needs of the tank industry, but the term “cylindrical wall” is more generally used.

3.1.37

shell-roof junction

junction between a cylindrical shell wall and the roof

Note 1 to entry: Alternatively known as the top angle or eaves junction, is the junction between the vertical wall and the roof.

3.1.38

sketch plate or bottom plate

plating used to form the internal bottom of a ground-supported tank

3.1.39

strake

single row of plates of a given thickness

Note 1 to entry: The cylindrical shell wall of a tank is formed by making horizontal circumferential joints between a group of short cylindrical sections, each termed a strake, formed by making vertical joints between individual curved plates. Several strakes normally form one course (see 3.1.7).

3.1.40

tank

vessel for storing liquid products

Note 1 to entry: In this standard it is assumed to be circular in plan.

3.1.41

Tank Group

TG

classification of a tank to identify the sophistication of its design requirements, according to its size, form and usage, placing it into Tank Group 0, 1, 2 or 3

Note 1 to entry: Tanks in Consequence Class 4 are in Tank Group 4. For these tanks additional considerations are necessary.

Note 2 to entry: The provisions of this standard are not required for TG0. All the provisions are intended to apply to TGs 1, 2 and 3, except where exemptions are specifically made for TG1 or TG2.

3.1.42

shell wall

metal plate elements forming the vertical walls, roof or a hopper bottom of a tank

Note 1 to entry: This term is not restricted to the vertical walls.

3.1.43

Structural Complexity Class

SCC

classification of a tank to address the complexity of the structural form, insofar as is necessary to meet the susceptibility to different failure modes

3.1.44

test pressure

pressure in the space above the test liquid during the test procedure

3.1.45

transition junction

junction between the vertical shell wall and a conical hopper

Note 1 to entry: The junction can be at the base of the vertical wall or part way down it.

3.1.46

unsupported shell roof structure

shell roof that has no truss or rafter structural framework beneath it, but relies on the resistance of the shell alone to carry loads and transfer them to the cylindrical wall of the tank

3.1.47

vertical stiffener

stiffener attached to the cylindrical shell wall in the axial direction

Note 1 to entry: It is sometimes used on elevated tanks to enhance the axial buckling resistance.

3.1.48

wind girder

substantial primary ring near the top of the cylindrical shell to provide both stiffness against buckling and strength against induced stresses under wind loading

## Symbols

For the purposes of this document, the following symbols apply.

The symbols used are based on ISO 3898.

### Roman upper-case letters

*A* area of structural member cross-section;

*A*1, *A*2 area of top, bottom flange of roof centre ring;

*E* Young’s modulus of elasticity;

*E*red reduced elastic modulus to account for thermal effects;

*F* force;

*FA* maximum vertical downward design distributed load on a roof including the weight of the plating, supporting structure and external vertical loads;

*FB* maximum vertical downward design distributed load on a roof including the weight of the plating, supporting structure and external vertical loads;

*I* second moment of area of cross-section;

*Ka* reduction factor for the effect of axial compression on buckling under external pressure;

*M* bending moment in a structural member;

*N* axial force in a structural member;

*N*f minimum number of load cycles relevant for fatigue;

*Pr* vertical load on a roof rafter;

*Rcr* linear elastic buckling resistance (see EN 1993‑1‑6)

*T* temperature;

*T*Ed reference temperature for design;

*T*LODMAT lowest one-day mean ambient temperature (see 5.4.2);

*T*MDMT minimum design metal temperature (see 5.4.2);

*W* elastic section modulus.

### Roman lower-case letters

*a* is the horizontal side length of a rectangular opening in a cylindrical shell wall;

*a*j vertical distance from secondary ring *j* to the next secondary ring (*j*+1) below it, or else where there is no lower secondary ring, to the bottom of the cylindrical wall;

*b* is the vertical height of a rectangular opening in a cylindrical shell wall;

*bk* is the radial width of the centre ring in a roof;

*b*r is the local distance between two rafters or purlins when designing a roof (nonlinear theory);

*c*p coefficient for wind pressure loading;

*d* diameter of cylindrical shell wall of a tank;

*dn* diameter of reinforcement around a manhole or nozzle;

*do* diameter of opening of a manhole or nozzle;

*e* distance of outer fibre of beam to beam axis;

*fr* replacement factor at a nozzle used in design of reinforced openings;

*fs* stress concentration factor at a nozzle;

*f*q axial buckling resistance reduction factor for fabrication quality class;

*f*yd design value of yield strength of steel;

*f*u ultimate strength of steel;

*hCO* rise of roof (height of apex of a conical roof above the plane of its junction to a cylindrical wall), see Figure 3.1 and 8.1;

*hS* rise of roof (height of apex of a dome or dished end above the plane of its junction to a cylindrical wall), see Figure 3.1 and 8.1;

*hb* height of an unstiffened cylindrical shell that can be stable against buckling measured from the top of the cylindrical wall, curb angle or primary wind girder (as appropriate);

*hbm* height of a potential buckle extending down from the curb angle or primary wind girder down to the base of the *m*th course (*i* = 1 for the top course);

*hCO* rise of roof (height of apex of a conical roof above the plane of its junction to a cylindrical wall), see Figures 3.1 and 8.1;

*hF* freeboard or height of the top of the shell above the maximum design liquid level (MDLL), see Figure 3.1;

*hg* height above the base of the centre of mass of the liquid in a pedestal tank (Figure 3.2);

*hi* depth of the base of the *i*th course below the top of a cylindrical shell wall (*i* = 1 at the top);

*hk* vertical distance between the flanges of the centre ring (Figure 8.2);

*hL* liquid level above the base in the considered design situation;

*hPR* height of the primary ring / primary wind girder from the base, see Figure 3.1;

*hrj* depth to the *j*th secondary ring below the primary ring (Figure 7.5);

*hS* rise of roof (height of apex of a dome or dished end above the plane of its junction to a cylindrical wall), see Figures 3.1 and 8.1;

*hT* height of the cylindrical shell from the base to the top of the cylindrical wall (Figure 3.1);

*hV* height of cylindrical wall of thickness *t*i needed to resist the internal vapour pressure (transformed into equivalent liquid); *i* the assigned number to a shell wall course, measured from the top (roof or wind girder);

*j* the assigned number to a secondary ring, measured from the top;

*jef* joint efficiency factor;

*ka* coefficient for axial compression in buckling resistance under external pressure;

*kw* coefficient for buckling resistance under wind;

*lS* height of shell segment or stiffener shear length;

*m* bending moment per unit width of plate;

*m* the number of courses in a group being assessed for buckling under external pressure;

*n* membrane stress resultant per unit width of plate;

*n* number of rafters in circular tank roof;

*p* distributed loading of any kind (not necessarily normal to the wall);

*pn* internal pressure normal to a tank wall surface (acting outward);

*pV,Ed* design value of the vapour pressure above the liquid surface;

*q* external pressure or wind pressure normal to a tank wall or roof (acting inward);

*r* radial coordinate of any point;

*r*CO radius of curvature of a conical roof (see Formula (8.22));

*r*S radius of curvature of a spherical dome roof;

*rT* radius of middle surface of a cylindrical tank wall (shell);

*t* shell wall thickness;

*tAP* thickness of the annular plate or base ring;

*tBP* thickness of the bottom plate, sketch plate or bottom central plate;

*tCO* thickness of the roof plate of a conical roof;

teq,m equivalent thickness of a potential external pressure buckle;

*ti* shell wall thickness in the ith course below the top of the tank wall;

*tS* thickness of the roof plate of a spherical dome roof;

*wg* size parameter used in defining the Structural Complexity Class;

*wAP* width of annular plate or base ring;

*wAPlim* minimum width of annular plate or base ring;

*wAP,i,min* minimum exposed inside width (distance from the inner edge of the annular base plate to the inner edge of the shell plate);

*x* vertical coordinate of any point in a tank;

*xCO* local vertical coordinate of a point on a conical tank roof (Figure 8.1);

*xL* global vertical coordinate measured down from the liquid surface level hL.

*xS* local vertical coordinate of a point on a spherical tank roof (Figure 8.1);

*xT* local vertical coordinate measured from the top of shell downwards (Figure 3.1);

### Greek letters

*α* slope of the outer edge of a roof relative to the horizontal (see Figures 3.1 and 8.1);

*α*θI imperfection reduction factor under external pressure or wind;

*β* inclination of a conical segment relative to the vertical (see Figure 13.1);

*β*r rafter spacing parameter = π/*n* where *n* is the number of rafters;

*γ*U upper characteristic value of the unit weight (specific weight) of a liquid;

*γ*F partial factor for actions (see EN 1990:2023, Clause A.4);

*γ*FL partial factor for liquid pressure actions;

*γ*FV partial factor for vapour pressure actions;

*γ*M partial factor for resistance;

*δ* deflection;

*η* dimensionless size of a circular opening in a cylindrical shell;

Δ change in a variable;

*ν* Poisson’s ratio;

*ρ* upper characteristic value of the mass density of a liquid;

*θ* circumferential coordinate around a shell;

*σ* direct stress;

*τ* shear stress.

### Subscripts

E value of stress or displacement (arising from design actions);

F at half span;

F action;

a annular;

d design value;

f fatigue;

i inside;

i inward directed;

i counting variable;

i the course number, counting from the primary ring or curb angle or roof;

j the secondary ring number, counting from the primary ring or roof to shell junction;

k roof centre ring;

k characteristic value;

m mean value;

min minimum allowed value;

n nominal;

n normal to the wall;

o outside;

o outward directed;

p pressure;

r radial;

r ring;

R resistance;

s at support;

s shell wall;

T tank cylindrical shell wall;

x axial (meridional in a cylinder);

θ circumferential;

z transverse;

y yield;

0 reference value;

1 upper;

2 lower;

θ circumferential;

ϕ meridional.

## Sign conventions

### Conventions for global tank structure axis system

(1) The sign convention given here is for the complete circular tank structure, and recognizes that the tank is not a structural member. Care with coordinate systems is required to ensure that local coordinates associated with members attached to the shell wall and loadings given in local coordinate directions but defined by a global coordinate are not confused.

(2) In general, the convention for the global tank structure axis system is in cylindrical coordinates (see Figure 3.1b) as follows:

Coordinate system

Coordinate on the central axis of a shell of revolution *x*

Radial coordinate *r*

Circumferential coordinate *θ*

(3) The convention for positive directions is:

Outward direction positive (internal pressure positive, outward displacements positive)

Tensile stresses positive (except in buckling equations where compression is positive)

(4) The convention for distributed actions on the tank wall surface is:

— Pressure normal to surface (outward pressure positive) *p*

— Pressure normal to surface (inward pressure positive) *q*

|  |  |
| --- | --- |
|  |  |
| **a) Ground-supported cylindrical tank with spherical dome roof, showing important levels** | **b) Vertical section through circular tank with conical roof: coordinates and loading** |

Key

|  |  |
| --- | --- |
| a | planform diameter |
| 1 | roof |
| 2 | cylindrical shell |
| 3 | annular plate |
| 4 | bottom |

Figure 3.1 — Circular tanks: general descriptors

NOTE For simplicity and clarity in Figure 3.1, only the liquid level MDLL has been identified by its dimensional location with the corresponding coordinate *x*L. The other defined liquid levels MNOL and min. NOL are marked but their locations are not identified.

### Conventions for structural element axes in circular tanks

(1) The notation and sign convention for ring stiffeners and wind girders attached to the tank wall (Figure 3.2) is:

— Circumferential coordinate axis (curved) *θ*

— Bending moments *M*θ and section modulus *I*θ for circumferential stresses *σ*θ

Figure 3.2 — Local coordinate system for ring stiffeners, wind girders or circumferential stiffeners on a cylindrical shell

### Conventions for stress resultants for circular tanks

(1) The convention used for subscripts indicating membrane forces is:

“The subscript derives from the direction in which direct stress is induced by the force” for direct stress resultants. For membrane shears and twisting moments, the sign convention is shown in Figure 3.3.

Membrane stress resultants:

*n*x axial membrane stress resultant in a cylindrical shell

*n*ϕ meridional membrane stress resultant in a conical or spherical shell

*n*θ circumferential membrane stress resultant in shells

*n*xθ membrane shear stress resultant in a cylindrical shell

*n*ϕθ membrane shear stress resultant in a conical or spherical shell

Membrane stresses:

*σ*mx axial membrane stress in a cylindrical shell

*σ*mϕ meridional membrane stress in a conical or spherical shell

*σ*mθ circumferential membrane stress in shells

*τ*mxθ membrane shear stress in a cylindrical shell

*τ*mϕθ membrane shear stress in a conical or spherical shell

(2) The convention used for subscripts indicating moments is:

The subscript derives from the direction in which direct stress is induced by the moment. For twisting moments, the sign convention is shown in Figure 3.3.

NOTE 1 This plate and shell convention is at variance with beam and column conventions used in EN 1993‑1‑1 and EN 1993‑1‑3. Care is needed when using them in conjunction with these provisions.

NOTE 2 To match bending terminology for shell structures, the vertical stresses in both the shell and any vertical stiffener are denoted σx. The bending moments inducing vertical stresses in a vertical stiffener are denoted as *M*x, and the second moment of area of a stiffener bending in the vertical direction (stresses in the vertical direction) (Figure 3.3a) is denoted as *I*x (not *I*θ as would be the case for a structural member according to EN 1993‑1‑1). Similarly, the circumferential stresses in both the shell and any ring stiffener are denoted *σ*θ. The bending moments inducing circumferential stresses in a ring stiffener are denoted as *M*θ, and the second moment of area of a ring (Figure 3.2) bending in the plane of the ring (stresses in the circumferential direction) is denoted by *I*θ. This last notation is important for the required stiffness of ring stiffeners to assist in resisting buckling under external pressure or wind.

Bending stress resultants:

*m*x axial bending moment per unit width in a cylindrical shell

*m*ϕ meridional bending moment per unit width in a conical or spherical shell

*m*θ circumferential bending moment per unit width in shells

*m*xθ twisting shear moment per unit width in a cylindrical shell

*m*ϕθ twisting shear moment per unit width in a conical or spherical shell

Bending stresses:

*σ*bx axial bending stress in a cylindrical shell

*σ*bϕ meridional bending stress in a conical or spherical shell

*σ*bθ circumferential bending stress in shells

*τ*bxθ twisting shear stress in a cylindrical shell

*τ*bϕθ twisting shear stress in a conical or spherical shell

Inner and outer surface stresses:

*σ*six, *σ*sox axial inner, outer surface stress

*σ*siϕ, *σ*soϕ meridional inner, outer surface stress

*σ*siθ, *σ*soθ circumferential inner, outer surface stress in shells

|  |  |
| --- | --- |
|  |  |
| **a) Membrane stress resultants** | **b) Bending stress resultants** |

Figure 3.3 — Stress resultants in a cylindrical shell wall

NOTE The shear and twisting arrows in Figure 3.3 do not follow the strict mechanics notation rules, because *n*θx and *m*θx are never used in EN 1993‑1‑6 or EN 1993‑4‑2, nor do the directions of these stress resultants every make any difference to the calculation rules. To show a distinction between *n*xθ and *n*θx or between *m*xθ and *m*θx would be misleading.

# Basis of design

## Basic requirements

(1) The design of steel tanks shall be in accordance with the general rules given in EN 1990 and the EN 1991 series and the specific design provisions for steel structures given in the EN 1993‑1 series.

(2) Tanks designed according to this document shall be executed according to EN 1090‑2 with construction materials and products used as specified in the relevant parts of EN 1993, or in the relevant material and product specifications.

(3) The structural design of the tank structure should consider all shell and plated sections of the structure, including any stiffeners, ribs, rings and attachments.

(4) Where the buckling resistance is critical, the fabrication tolerances defined in EN 1993‑1‑6 should be adopted.

NOTE The tolerance measurement used in EN 1993‑1‑6 for circumferential variations in the tank wall can be performed using either a physical template or another appropriate measurement such as a laser scan.

(5) The design of a supporting structure for a tank is covered by the provisions of EN 1993‑1‑1 for steel structures and of EN 1992‑1‑1 for concrete structures.

(6) Any supporting structure should not be treated as part of the tank. Similarly, other structures supported by the tank should be treated as beginning where the tank wall or attachment ends. However, interactions between the supporting structure and the tank (e.g. fatigue induced into the supporting structure due to varying tank loads, seismic interactions, etc.) should be considered.

(7) The design of ancillary structures such as stairways, platforms, pipe racks and ladders is covered by the applicable rules in EN 1993‑1‑1 and EN 1992‑1‑1.

(8) The design of an aluminium roof structure on a steel tank is covered by the provisions of EN 1999‑1‑5.

(9) The design of a conical hopper for a tank is covered in the provisions of EN 1993‑4‑1.

(10) The design of a transition junction between the base of a cylindrical shell wall and a conical hopper, and the design of a supporting ring girder in an elevated tank are covered by the provisions of EN 1993‑4‑1.

(11) Reinforced concrete foundations for steel tanks are covered by the provisions of the EN 1992 series and the EN 1997 series.

(12) Numerical values of the specific actions on steel tanks are given in EN 1991‑4. The partial factors for actions are given in EN 1990:2023, Clause A.4.

(13) Tanks should be designed to be damage-tolerant where appropriate, considering the intended use of the tank.

(14) Particular requirements for special applications may be specified by the relevant authority, or where not specified, may be agreed for a specific project by the relevant parties.

(15) Special consideration should be given to situations that can occur during execution.

NOTE Other design requirements are defined in other Eurocode parts (EN 1990 to EN 1999) including EN 1997 for foundations and settlement as well as EN 1090‑2 for fabrication and execution. Other useful information is given for ambient temperature tanks in EN 14015 and for RLG tanks in EN 14620‑2. The information in these two standards also relates to special issues on foundations and settlement, fabrication, execution, testing, functional performance, and details like man-holes, flanges, and filling devices.

(16) For ambient temperature tanks, fabrication and execution of flat bottomed ground-supported tanks also complies with the additional relevant requirements of EN 14015 where they are not contradictory to those of the Eurocodes and EN 1090‑2.

## Units

(1) For the application of this standard S.I. units are used in accordance with ISO 1000.

(2) For calculations, the following two alternative consistent units are recommended:

— Dimensions: m mm

— unit weight: kN/m3 N/mm3

— forces and loads: kN N

— line forces and line loads: kN/m N/mm

— pressures and area distributed actions: kPa MPa (=N/mm2)

— unit mass: t/m3 t/mm3

— acceleration: m/s2 mm/s2

— membrane stress resultants: kN/m N/mm

— bending stress resultants: kNm/m Nmm/mm

— stresses and elastic moduli: kPa MPa (=N/mm2)

NOTE 1 These two alternative consistent sets of units are given here because tank designers have often previously used formulae that include mixed inconsistent units that were specified for each formula. All formulae in this document are given without units. The notation t signifies a metric tonne = 1 000 kg = 1 Mg.

NOTE 2 When dynamic calculations are necessary, the strict application of S.I. units is recommended.

## Tank classification

### Reliability differentiation

(1) For reliability differentiation, see EN 1990:2023, Clause A.4.

(2) The classification of a tank into a Consequence Class is given in EN 1990:2023, Clause A.4, based on the consequences of failure. The classification for structural design rules combines the Consequence Class with the Structural Complexity Class into a single classification into a Tank Group.

### Structural complexity classification for tanks

(1) The Structural Complexity Class for a tank shall be determined by the conditions of the individual tank.

(2) The dimensions of the tank should be used as a critical part of the evaluation of the Structural Complexity Class.

(3) The effect of tank size on the selection the Structural Complexity Class should be determined using the parameter *w*g, calculated from Formula (4.1):

 (4.1)

where

|  |  |
| --- | --- |
| *d* | is the tank diameter (Figure 4.1); |
| *h*g | is the height of the centre of mass of the stored liquid above the ground (see Figure 4.1) at the maximum normal operating liquid level (MNOL) (see 3.1.23). |

NOTE 1 Quantified limits of the parameter *w*g for each Structural Complexity Class for tanks are given in Table 4.1 (NDP), unless the National Annex gives different quantified limits.

NOTE 2 The parameter *w*g provides a simple understandable single dimension that is related to the potential energy of the stored liquid, though the liquid density is omitted.

|  |  |
| --- | --- |
|  |  |
| **a) Height of the centre of mass of the liquid in ground-supported tank** | **b) Height of the centre of mass of the liquid in elevated tanks** |

Figure 4.1 — Dimensions defined for structural complexity class

Table 4.1 (NDP) — Descriptions and quantified limits for parameter *w*g for Structural Complexity Classes for tanks

| **Structural Complexity Class** | **Descriptions and parameter** *w*g **limits** |
| --- | --- |
| Structural Complexity Class 3 | a) Elevated tanks on discrete supports with *wg* ≥ 30 mb) Distillation tower tanks |
| Structural Complexity Class 2 | All other tanks within the scope of this standard |
| Structural Complexity Class 1 | Tanks with 2 < *wg* ≤ 15 m |
| Structural Complexity Class 0 | Tanks with *wg* ≤ 2 m |

### Tank group classification

(1) Different levels of rigour should be used in the design of tank structures, depending on the Tank Group to which the tank belongs, the structural arrangement and the susceptibility to different failure modes.

(2) Classification into a Tank Group should be determined by the conditions of the individual tank.

NOTE The National Annex can define the conditions to distinguish between Tank Groups on the basis of the location, the size, the stored liquid and loading, the structural form, and operational aspects (see EN 1990:2023, Clause A.4 and EN 1991‑4). It can also choose appropriate values for the boundaries between the classes.

(3) Four Tank Groups are used in this standard, with requirements that produce designs with essentially comparable demands on the design assessment and considering the expense and procedures necessary to ensure the safety of different structures: Tank Groups 0, 1, 2 and 3 as indicated in Table 4.2.

NOTE Tanks that could be classified as Tank Group 4 correspond to structures in Consequence Class 4 (EN 1990), which are outside the scope of the Eurocodes since additional information is required for their design.

(4) The structural design should be carried out according to the provisions for the relevant Tank Group given in this part.

(5) The provisions for a higher Tank Group than that required can always be adopted for the design.

(6) The Tank Group should be chosen as specified by the relevant authority or, where not specified, agreed for a specific project by the relevant parties.

NOTE The classification of a tank into a Tank Group is given in Table 4.2, unless a different choice has been agreed as defined in (6).

Table 4.2 — Tank Group (TG) chosen according to the Structural Complexity Class (SCC), and Consequence Class (CC)

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Structural Complexity Class (SCC)** | **Higher** | **Normal** | **Lower** | **Lowest** |
| **(CC3)** | **(CC2)** | **(CC1)** | **(CC0)** |
| Higher (SCC3) | TG3 | TG3 | TG2 | TG1 |
| Normal (SCC2) | TG3 | TG2 | TG2 | TG0 |
| Lower (SCC1) | TG3 | TG2 | TG1 | TG0 |
| Lowest (SCC0) | TG2 | TG2 | TG1 | TG0 |

##  Verification by the partial factor method

### Partial factors for actions on tanks

(1) For persistent, transient and accidental design situations, the partial factors *γ*F shall be taken from EN 1990:2023, Clause A.4.

(2) The general requirements of EN 1990 should be satisfied.

(3) The recommended values of the partial factors for liquid actions on tanks are defined in EN 1990:2023, Clause A.4.

NOTE The values of partial factors *γ*F for tanks can be defined in the National Annex. Recommended values for the liquid loads are given in EN 1990:2023, Clause A.4.

### Partial factors for resistances

(1) Where structural properties are determined by testing, the requirements and procedures of EN 1990 should be adopted.

(2) Fatigue verifications should satisfy Clause 10 of prEN 1993‑1‑6:2023 and EN 1993‑1‑9.

(3) The partial factors *γ*Mi for different limit states shall be taken from Table 4.3.

Table 4.3 — Partial factors for resistance for tanks

|  |  |
| --- | --- |
| **Resistance to failure mode** | **Relevant *γ*** |
| Resistance of welded or bolted wall plastic failure | *γ*M0 |
| Resistance of shell wall to stability | *γ*M1 |
| Resistance of bolted wall to rupture | *γ*M2 |
| Resistance to cyclic plasticity | *γ*M4 |
| Resistance to fatigue | *γ*Mf |

(4) The numerical values for partial factors for resistance for tanks in Table 4.4 are recommended.

NOTE 1 The numerical values for partial factors for resistance for tanks are given in Table 4.4, unless the National Annex gives different values.

Table 4.4 (NDP) — Numerical values for partial factors for resistance for tanks

|  |  |  |
| --- | --- | --- |
| *γ*M0 = 1,1 | *γ*M1 = 1,1 | *γ*M2 = 1,25 |
|   | *γ*M4 = 1,00 | *γ*Mf see EN 1993‑1‑9 |

NOTE 2 Some numerical values given in Table 4.4 differ from those in EN 1993‑1‑1 because the entire tank cylindrical shell is under membrane tension and a failure here would be catastrophic. By contrast, the loading is very well defined so EN 1990:2023, Clause A.4 does not require a high partial factor on actions.

(4) Where hot-rolled steel sections are used as part of a tank structure (e.g. an elevated platform on the tank roof or a truss supporting the roof), the relevant partial factors for their resistance should be taken from EN 1993‑1‑1.

(5) Safety factors for ‘product type’ tanks (factory production) can be specified by the appropriate authorities. When applied to ‘product type’ tanks, the factors here are for guidance purposes only. They are provided to show the levels likely to be needed to achieve consistent reliability with other designs.

### Serviceability limit states

(1) Where simplified compliance rules are given in the relevant provisions dealing with serviceability limit states, detailed calculations using combinations of actions may be omitted.

(2) For all serviceability limit states the values of *γ*Mser should be specified.

NOTE The value *γ*Mser = 1,0, unless the National Annex provides a different value.

## Limit states

(1) The limit states defined in EN 1993‑1‑6 should be adopted for tanks.

— Global stability and static equilibrium as a rigid body;

— LS1: plastic limit;

— LS2: cyclic plasticity

— LS3: buckling;

— LS4: fatigue.

## Actions and environmental effects

(1) The general requirements set out in Clause 4 of EN 1990:2023 shall be satisfied.

## Material properties

(1) The general requirements for material properties given in EN 1993‑1‑1, EN 1993‑1‑10 and EN 1993‑1‑4, as appropriate, should be followed, except as defined in Clause 5.

(2) The specific properties of materials for tanks given in Clause 5 of this document should be used.

(3) Due to the risk of stress corrosion cracking, particular attention should be paid to the choice of steel for tanks and structures used to store ammonia. This also applies in particular to the welding consumables. In addition, the welding process, the post-weld treatment and production control must be adjusted accordingly.

(4) The choice of steel should be adapted to the storage medium and should be compatible with the stored liquid to avoid degenerative effects.

## Geometrical data

(1) The general information on geometrical data provided in EN 1990 may be used.

(2) Additional information specific to shell structures (e.g. the curvature of surfaces, the inward or outward eccentricity of a stiffener, the properties of rings) provided in EN 1993‑1‑6 may be used.

(3) The plate thicknesses given in 6.1.2 should be used in calculations.

## Modelling of the tank for determining action effects

(1) The general requirements of EN 1990 shall be followed.

(2) The specific requirements for structural analysis in relation to serviceability set out in Clause 14 should be used for the relevant structural segments.

(3) The specific requirements for structural analysis in relation to ultimate limit states set out in 6.2 (and in more detail in EN 1993‑1‑6) should be applied.

## Design assisted by testing

(1) The general requirements set out in Annex D of EN 1990:2023 should be followed.

## Durability

(1) The general requirements for durability set out in EN 1990 shall be followed.

(2) The responsibility for corrosion losses in a tank lies with the client, owner or end user.

(3) The life expectancy of the tank and its intended usage should be agreed between the client, the engineer and the relevant authority.

(4) The extent of corrosion loss depends upon the stored liquid, the type of steel, the heat treatment, the life expectancy of the tank and any measures taken to protect the construction against corrosion.

NOTE The National Annex can choose appropriate values for corrosion losses for particular liquids in contact with defined tank materials for a defined life expectancy.

(5) Where a suitable protection system, as approved by the relevant authority where appropriate, is provided to guarantee protection against corrosion (e.g. glass lining or epoxy coating or galvanised internal surface, cathodic protection, etc.), no corrosion loss provision need be considered.

(6) The thickness reduction to account for the effects of corrosion of the shell wall and all other parts of the tank should be agreed between the designer, the client and the relevant authority, taking account of the intended use, any internal lining, the nature of the liquid to be stored and the life expectancy of the tank. Different reductions may be assigned to different parts of the tank.

NOTE Further useful information is available in EN 12285‑1, where it is applicable.

(7) Consideration should also be given to corrosion by the atmosphere above the level of the stored liquid, especially if it can contain corrosive vapour such as water vapour.

(8) Wall thickness losses and damage to internal structures due to corrosion should be considered in the design calculations.

(9) Loss of thickness in the tank base plate due to corrosion should be considered in the design.

(10) Guidance for the selection of stainless steels in view of corrosion actions may be obtained from EN 1993‑1‑4.

(11) Appropriate provision should be made for periodic inspection of the tank wall thickness, with reference made to its original design thickness at every level.

# Properties of materials

## General

(1) All steels used for tanks should be suitable for welding to permit later modifications when necessary.

(2) All steels used for tanks of circular planform should be suitable for cold forming into curved sheets or curved members.

(3) The material properties given in any relevant reference standard should be treated as nominal values to be adopted as characteristic values in design calculations.

(4) Where it is possible that the tank will be filled with a hot liquid, the design values to be used should be appropriately reduced to material property values corresponding to the maximum temperature to be expected (see 5.4.2). Special attention should be paid to the decrease in the elastic modulus at elevated temperatures, since many buckling modes in tanks occur under elastic conditions (see EN 1993‑1‑6 and EN 1993‑1‑2).

(5) The methods for design by calculation given in this document may be used for all structural steels as defined within any part of EN 1993 provided that the ratio *f*u/*f*y is not less than 1,05.

## Structural carbon and carbon manganese steels

(1) The selection of carbon or carbon manganese steels for a project should be agreed between the designer, the client and the relevant authority.

NOTE Useful lists can be found in EN 14015 and EN 14620‑2.

(2) The mechanical properties should be according to EN 10025 series, EN 10149 series, EN 10216 series, EN 10217 series and EN 10028 series and should be taken from EN 1993‑1‑1, EN 1993‑1‑3 and EN 1993‑1‑12 as appropriate, except as defined in (3).

(3) For the steels covered by this document, the design value of Poisson’s ratio should be taken as *ν* = 0,3. The characteristic value of the elastic modulus for structural steel should be taken as *E* = 200 000 N/mm2.

NOTE Elastic buckling can be critical in tank structures, so a conservative estimate of the elastic modulus is made, since the partial factor for buckling *γ*M1 has been made consistent throughout all the EN 1993 standards.

(4) Ambient temperature tanks constructed using carbon or carbon manganese steel grades may be designed according to the rules of this standard if the temperature to which they are exposed in service lies within the range –50 °C ≤ *T* ≤ +300 °C. For lower temperatures, see (5). Where a tank using these materials is susceptible to failure by fatigue, the temperature should be limited to *T* ≤ 150 °C.

NOTE Temperatures referred to in these provisions relate to the operating temperature of the tank.

(5) Where a tank can be subject to operational temperatures below −50°C, the brittle fracture and toughness requirements for the material should be addressed separately using relevant materials data. Suitable provisions are given in EN 14620‑2.

(6) The material properties of steels at temperatures above 50 °C should be obtained from EN 1993‑1‑2. Where these are adopted, the reduced yield stress may be conservatively taken as the temperature-dependent proportionality limit *f*pθ. The mechanical properties for steel grades not represented in EN 1993‑1‑2 should be based on reliable information.

(7) Where the design involves a stability calculation, appropriate reduced properties should be used either as appropriate to the operating temperature (see EN 1993‑1‑2) or as appropriate to the stress-strain curve where higher stresses are involved (see EN 1993‑1‑14). The reduced elastic modulus *E*red should be adopted.

## Structural stainless steels

(1) The selection of stainless steels for a project should be agreed between the designer, the client and the relevant authority.

NOTE Useful lists can be found in EN 14015 and EN 14620‑2.

(2) The mechanical properties of stainless steel according to EN 10088‑4, EN 10088‑5, EN 10028‑7, EN 10216‑5, EN 10217‑7, EN 10272 and EN 10222 series should be taken from EN 1993‑1‑4.

(3) For ambient temperature tanks constructed using stainless steel, the following temperature ranges are accommodated within the rules of this standard:

i. tanks constructed using austenitic stainless steels, –50 °C ≤ *T* ≤ +300 °C;

ii. tanks constructed with special austenitic grades that have defined yield strengths up to higher temperatures, –50 °C ≤ *T* ≤ the maximum defined temperature for the grade;

iii. tanks constructed using austenitic-ferritic stainless steels, –50 °C < *T* < +250 °C;

iv. tanks constructed using ferritic stainless steels, –10 °C < *T* < +250 °C.

NOTE For refrigerated liquefied gas (RLG) tanks, see EN 14620‑2.

(4) The material characteristics at elevated temperatures *T* ≥ 50 °C should be obtained from EN 10088‑4 and EN 10088‑5, or from EN 10028‑7.

(5) Where buckling controls the resistance at a temperature above ambient, the reduced values of modulus at elevated temperature should be adopted according to the provisions of EN 1993‑1‑4.

## Toughness requirements

### General

(1) The fracture toughness requirements should be determined for the reference temperature *T*ed according to EN 1993‑1‑10 for carbon steels and EN 1993‑1‑4 for stainless steels.

(2) The minimum design metal temperature *T*MDMT should be determined according to 5.4.2. The temperature *T*MDMT should be used in place of (*T*MD + Δ*T*r) as defined in EN 1993‑1‑10.

### Minimum design metal temperature

(1) The minimum design metal temperature *T*MDMT should be taken as the lowest of the minimum temperature of the contents and the climatic temperatures defined in Table 5.1 and EN 1991‑1‑5.

(2) The lowest one-day mean ambient temperature *T*LODMAT should be taken as the lowest recorded temperature averaged over any 24 h period. Where insufficiently complete records are available, this average temperature may be taken as the mean of the maximum and minimum temperatures or an equivalent value.

NOTE 1 The identification of the relevant value for *T*LODMAT can require consideration of the both the relevant return period, and the temperature of the stored product.

NOTE 2 Due to the evaporation of liquid during the commissioning of a tank, the tank can be exposed for a short time to temperatures lower than the storage temperature.

NOTE 3 Further useful information can be found in EN 1991‑1‑5, where the relevant temperature *T*LODMAT can be assessed for probabilites other than *P* = 0,02. In particular, the minimum winter ambient temperature based on 50 years and that for a longer period (100 or 200 years) can also be simply identified for a specific location.

Table 5.1 — Minimum design metal temperature *T*MDMT based on *T*LODMAT

|  |  |
| --- | --- |
| **Lowest one-day mean ambient temperature *T*LODMAT** | **Minimum design metal temperature *T*MDMT** |
|  | **10 years of data** | **30 years of data** |
| −10°C ≤ *T*LODMAT | *T*LODMAT +5°C | *T*LODMAT +10°C |
| −25°C ≤ *T*LODMAT ≤ −10°C | *T*LODMAT | *T*LODMAT +5°C |
| *T*LODMAT ≤ −25°C | *T*LODMAT −5°C | *T*LODMAT |

# Basis for structural analysis

## Ultimate limit states

### Basis

(1) Steel structures and components should be so proportioned that the basic design requirements given in Clause 4 are satisfied.

### Plate thickness to be used in resistance calculations

(1) In calculations to determine the resistance, the design thickness for a plate should be taken as Class A according to EN 10029 considering the maximum minus tolerance, unless a different class has been agreed between designer and client for a specific project.

(2) The design plate thickness should be reduced by a value of corrosion allowance as defined in 4.10.

### Fatigue

(1) Where many full cycles of filling and emptying of the tank are anticipated, the structure should be checked against the fatigue limit state (see EN 1993‑1‑6 and EN 1993‑1‑9). The manner in which the cycles should be counted is defined in prEN 1991‑4:2024, 11.6.

(2) The design against low cycle fatigue may be carried out according LS2 in EN 1993‑1‑6.

NOTE Additional guidance is given in [1] and [2].

(3) If variable actions will be applied with more than 10 000 cycles during the design life of the tank the design should be checked against fatigue (LS4) according to Clause 10 of prEN 1993‑1‑6:2023 and EN 1993‑1‑9.

### Allowance for temperature effects

(1) The effects of differential temperature between parts of the structure should be included in determining the stress distribution depending upon the ultimate limit state considered.

## Analysis of the circular cylindrical shell structure of a tank

### Modelling of the structural shell

(1) For ambient temperature tanks, the modelling of the entire structural shell should follow the requirements of EN 1993‑1‑6, but these may be deemed to be satisfied by the following provisions:

— The structural shell consists of cylindrical, conical, spherical or toroidal segments.

— The modelling of the structural shell includes all stiffeners, openings and attachments, unless, for each failure mode separately, it can be shown that these details do not weaken the structure.

— The design details ensure that the assumed boundary conditions are satisfied.

### Methods of analysis

#### General

(1) The analysis of the shell of the tank should be carried out according to the requirements of EN 1993‑1‑6.

(2) A higher class of analysis may always be used than that defined for the selected Tank Group.

(3) For all tank groups, computational analysis should be used for problems that cannot be reliably solved using analytical formulae.

NOTE Computational analysis is especially useful in treating the many load combinations required by EN 1990:2023, Clause A.4.

(4) Where the provisions of 6.2.2.3 and 6.2.2.4 require computational analysis, it should always be used.

#### Tank Group 1

(1) For tanks in Tank Group 1, membrane theory may be used to determine the primary stresses.

NOTE The secondary stresses due to local bending are ignored because the minor level of plasticity that occurs is not reversed and involves a permanent change of shape.

#### Tank Group 2

(1) For tanks in Tank Group 2 under axisymmetric actions and supports, one of two alternative analyses should be used:

a) membrane theory may be used to determine the primary stresses with factors and simplified formulae to describe local bending effects and unsymmetrical actions;

b) a validated computational analysis may be used, as defined in EN 1993‑1‑6, including the considerations defined in 6.2.2.4(5).

(2) Where the loading condition is not axisymmetric, a computational analysis should be used, except under the conditions set out in (3) and (4) below.

(3) Notwithstanding (2), where the loading varies smoothly around the shell causing global bending only (i.e. in the form of circumferential harmonic 1), membrane theory may be used to determine the three primary membrane stress resultants *n*x, *n*θ and *n*xθ.

(4) For analyses of actions due to wind loading and/or foundation settlement, semi-membrane theory or membrane theory may be used. The resistance to wind may be deemed to be met by the specific provisions for its assessment in Clause 7 without a detailed stress analysis of the stresses within the structure.

NOTE For information concerning the membrane theory and semi-membrane theory of shells, see EN 1993‑1‑6. The semi-membrane theory describes the behaviour of a cylindrical shell in which circumferential bending effects couple with vertical membrane stresses under unsymmetrical loads.

(5) Where membrane theory is used to analyse the shell, discrete rings attached to an isotropic cylindrical shell under internal pressure may be deemed to have an effective area which includes a length of shell above and below the ring of 0,78, except where the ring is at a junction or a change of plate thickness. Where the ring is at the top of the shell wall so that the ring is only connected to a shell segment on one side the contribution length of shell should be reduced to 0,39. A more precise treatment, including the effects of an additional contributing segments or thickness changes in the shell may be found using the provisions of Clause 10 of prEN 1993‑4‑1:2024 for transition junctions.

NOTE Where a ring has a significant stiffness to a uniform applied torque around the circumference, this can provide a small increase in the above effective lengths.

(6) Where the shell is discretely stiffened by vertical stiffeners, see EN 1993‑4‑1.

(7) Where a ring girder is used above discrete supports, compatibility of the axial deformation between the ring and adjacent shell segments should be considered. Where such a ring girder is used, the eccentricity of the ring girder centroid and shear centre relative to the shell wall and the support centreline should be included (see EN 1993‑4‑1). The geometric limitations defined in EN 1993‑4‑1 should be adopted.

(8) Where a ring girder is used to redistribute forces into discrete supports and bolts or discrete connectors are used to join the structural elements, the shear transmission between the ring parts due to shell and ring girder bending phenomena should be determined using an appropriate analysis (see EN 1993‑1‑6).

#### Tank Group 3

(1) Tanks in Tank Group 3 that have any special or complex details the internal forces and moments should be determined using a validated computational analysis as defined in EN 1993‑1‑6.

(2) Special or complex details, as indicated in (1), should be identified as:

i. unanchored tanks where uplift is possible under design conditions (see also 12.1);

ii. tanks with penetrations whose dimensions exceed whichever is the smallest of 0,1*H*S, 0,05*d* or 500 mm;

iii. adjacent to the termination of a discrete rib (see 3.1.30) that supports another structure;

iv. attachment to other structures;

v. penetrations classed as large in 11.3, that are not circular or have rib stiffeners;

vi. closely spaced penetrations and their surroundings;

vii. any other situation that has not been commonly used in previous constructions.

NOTE 1 Attachment to other structures can include:

a) platforms and stairs;

b) nozzles, mixers and instruments;

c) stiff structures and piping connecting a tank to other structures;

d) internal tank structures such as an inlet manifold or diffuser, or a heating coil;

e) structures that can apply unusual loads to the tank, especially if it is transverse to wall or roof.

NOTE 2 Computational analysis is not required where the penetration is treated according to the provisions of Clause 11.

(3) For features of tanks in Tank Group 3 that do not have complex details (see (2)), the requirements may be limited to those that apply to Tank Group 2 (see 6.2.2.3).

(4) The plastic limit state (LS1) may be assessed using plastic collapse strengths under primary stress states as defined in EN 1993‑1‑6.

(5) The outcome of validated computational analysis, as defined in EN 1993‑1‑6, should always be checked against analytical formulae, where possible. The following considerations should be taken into account:

— Computational analysis should be used for problems that cannot be reliably solved using analytical formulae (for all tank groups);

— In cases where reliable analytical formulae can be used (e.g. for major parts of the structure, such as the cylindrical wall under internal pressure), the results of computational analysis should not differ significantly from those from the analytical formulae;

— The computational modelling rules of EN 1993‑1‑14 should be used to verify the modelling and computational methodology.

NOTE These considerations also apply to tanks in Tank Group 2 where computational analysis has been used.

### Geometric imperfections and tolerances

(1) Geometric imperfections in the shell should satisfy the limitations defined in EN 1993‑1‑6 for design situations where the resistance is found to depend on the assumed tolerances.

NOTE 1 The requirements to meet geometric imperfection tolerances according to EN 1993‑1‑6 depend on the calculated stress state in the designed structure. Where buckling is not the controlling consideration, there is no requirement to meet these additional tolerances.

NOTE 2 The tolerance requirements for operational purposes can be found in EN 14015 or EN 14620‑2, as appropriate.

(2) Each tank should be defined as being in a specific Fabrication Tolerance Quality Class A, B, or C according to EN 1993‑1‑6 and EN 1090‑2.

(3) For tanks in Tank Groups 2 or 3, the geometric imperfections should be measured following hydro testing to ensure that the fabrication tolerance assumed in the design has been achieved.

(4) Shop fabricated bolted tanks in which hydro testing has been performed on site before final tightening of all bolted joints have very small negligible imperfections and may be considered to meet the local dimple requirements of Fabrication Tolerance Quality Class A according to EN 1993‑1‑6 without on-site verification.

(5) Geometric imperfections in the shell may be omitted in determining the internal forces and moments, except where a GNIA or GMNIA analysis is used, as defined in EN 1993‑1‑6.

## Tanks constructed using corrugated sheeting

(1) Where corrugated sheeting is used as part of the tank structure, the analysis may be carried out treating the sheeting as an equivalent orthotropic wall (see EN 1993‑4‑1).

(2) The orthotropic properties obtained from considering the load displacement behaviour of the corrugated section in the orthogonal directions may be used in a stress analysis and in a buckling analysis of the structure. The properties may be determined as described in 6.4 of prEN 1993‑4‑1:2024.

# Design of cylindrical shell walls

## Basis

### General

(1) Cylindrical shell walls should be so proportioned that the basic design requirements for the ultimate limit states given in Clause 4 are satisfied.

(2) The safety assessment of the cylindrical shell should be carried out using the provisions of EN 1993‑1‑6, but they may be deemed to be satisfied when the design of the cylindrical wall includes all load cases and load combinations according to EN 1990:2023, Clause A.4, including:

— self weight of the complete structure;

— internal pressure due to the stored liquid;

— positive or negative vapour pressure above the surface of the liquid;

— axial and radial forces due to imposed loads on roofs, platforms, insulation, etc.;

— environmental loads including wind, snow and ice, floods and earthquake;

— differential settlement of the foundation;

— local loads transferred to the shell from supports, platforms and piping (nozzles).

### Cylindrical shell wall design

(1) The cylindrical shell wall of the tank should be checked for the phenomena described in (2) to (9) under the limit states defined in EN 1993‑1‑6.

(2) The cylindrical shell wall should satisfy the provisions of EN 1993‑1‑6, except where this standard provides alternatives that are deemed to satisfy the requirements of that standard.

(3) Tanks in all Tank Groups should satisfy the requirements for LS1 (plastic limit) and LS3 (buckling).

(4) For tanks in Tank Group 1, the cyclic plasticity (LS2) and fatigue (LS4) limit states may be ignored.

(5) For tanks in Tank Group 2, the fatigue check may be ignored except where the design life of the structure involves more than 10 000 cycles of filling and emptying (see 6.1.3) or where sources of vibration can lead to significant local stresses with more than 10 000 cycles in the design life.

(6) For tanks in Tank Group 2, cyclic plasticity may be ignored, except where the tank will be subjected to more than 12 complete fillings and emptying per year (see 6.1.3).

NOTE The limit state of cyclic plasticity involves alternating yielding in a design situation. In tanks this limit state is unlikely to occur, except in anchor details or similar points producing local high stresses.

(7) For tanks in Tank Group 3, the limit states of cyclic plasticity (LS2) and fatigue (LS4) should be evaluated with special attention to anchorage stresses and local stress raisers.

(8) Where vibrating machinery (e.g. an agitator) is attached to the tank, or where shock waves can propagate in attached piping, the potential number of cycles of induced stress should be evaluated.

(9) For these alternating load conditions, a linear analysis (LA) according to EN 1993‑1‑6 should be performed. Where the number of cycles can exceed 10 000, the potential for fatigue failure should be assessed according to the provisions of EN 1993‑1‑6 (LS4) and EN 1993‑1‑9. Where the number of cycles is less than 10 000, but the loading induces high alternating stresses, the potential for cyclic plasticity should be assessed according to the provisions of EN 1993‑1‑6 (LS2).

### Catch basins

(1) The requirement for a catch basin and its construction type should be determined according to the liquid to be stored.

NOTE 1 Supplementary rules and requirements for specific liquids (e.g. water polluting liquids, spread and diffusion of liquid and vapours, etc.) can be defined in the National Annex.

NOTE 2 Consideration of the possibility of leakage or failure of the liquid containing tank (e.g. exposure to liquid temperature and thermal gradients, sudden vapour generation, etc.) are necessary for the design of the catch basin.

NOTE 3 Leakage or failure of the liquid containing tank should be treated as an accidental load case for the design of the catch basin.

(2) The leakage of liquid from a tank into a catch basin, with or without a roof, is considered an accidental design situation as per EN 1990.

(3) Open topped catch basins should be designed according to the provisions of this document.

(4) The wall of the catch basin should not be closer than 1m from the exterior of the tank cylindrical shell.

NOTE This provision ensures that there is sufficient space for inspection and maintenance work.

(5) The annular space between the tank and the catch basin wall should be designed in such a way that rain water and condensation or fire extinguishing water cannot collect in this space.

NOTE 1 This requirement prevents the tank shell from collapse under external hydrostatic pressure.

NOTE 2 The requirement can be achieved by the use of appropriate equipment (e.g. water pumps) in the annular space.

(6) The tank foundations and any structures that support the tank should be designed to accommodate the loads and load combinations that can act on the catch basin wall.

(7) If the catch basin has a roof, the rules for the design of tank roofs given in this document also apply to the catch basin roof.

## Resistance of the cylindrical shell

### General

(1) The resistance of the cylindrical shell should be evaluated using the provisions of EN 1993‑1‑6. However, the provisions of 7.3, 7.4 and 7.5 and 11 may be deemed to satisfy the provisions of that standard wherever appropriate rules are given.

(2) The requirements of EN 1090‑2 for execution should be met.

(3) The joint efficiency factor *j*ef for full penetration butt welds should be taken from EN 1993‑1‑6 where it is defined as *j*ef = 1,0. The relevant requirements of EN 14015 and EN 14620‑2 should also be met where they have stricter requirements on joint efficiency than the Eurocodes and EN 1090‑2.

(4) The joint efficiency factor *j*ef for lap welded joints should be taken from EN 1993‑1‑6.

(5) For bolted tanks, the joints should be designed for the governing failure mode to be net section failure. Where this is not possible, the governing failure mode of bolt shear is permitted (see EN 1993‑1‑8).

(6) For bolted joints, the joint efficiency *j*ef should be based on the governing resistance of the bolted connection relative to the yield strength of the parent plate. The governing resistance should be found from EN 1993‑1‑8. Care should be taken to identify the critical failure mode, and the resistance assessment should take into account all possible failure modes, including tension on the net or gross section, bolt shear and the bearing resistance of bolts (see EN 1993‑1‑8).

(7) For other types of connection, the joint design should be in accordance with EN 1993‑1‑8.

(8) Where the material of the tank is stainless steel, the provisions of EN 1993‑1‑4 should be adopted.

## Cylindrical shell plate thickness in a stepped-wall to resist liquid pressures

(1) The circumferential membrane stress due to liquid loads and internal vapour pressure should be verified in each shell course using the following procedure. This procedure should be used to address both the service load condition and the tank test condition (see EN 1991‑4).

(2) Two alternative methods may be used to verify the resistance of the wall, either as defined in (3) to (5), or by the direct method in (6) to (11).

(3) Where the plate thicknesses and steel grades at different heights in the tank have been already selected, they should be verified using the provisions of (4) and (5).

(4) The pressure against the wall at any level in the tank, allowing for the depth *z* below the surface, the design value of the unit weight of liquid and the design value of any additional positive pressure from vapour in the space above the surface, should be obtained as *p*n,Ed.

 (7.1)

where:

|  |  |
| --- | --- |
| *xL* | is the distance below the liquid surface level defined as at *h*L |
| *γ*FL | is the partial factor on liquid pressures (see EN 1990:2023, Annex A); |
| *γ*FV | is the partial factor on internal gas pressures (see EN 1990:2023, Annex A); |
| *γ*U | is the upper characteristic value of the unit weight (specific weight) of the stored liquid; |
| *p*V | is the characteristic value of the vapour pressure above the liquid surface. |

(5) The resistance of the wall at the base of every course against this internal pressure should be verified using;

 (7.2)

where:

|  |  |
| --- | --- |
| *γ*m0 | is the partial factor for plastic failure (see Table 4.4); |
| *p*n,Ed | is the local design value of the internal pressure acting on the wall; |
| *r*T | is the radius of the middle surface of the wall; |
| *ti* | is the local thickness of the wall in the ith course; |
| *Fyi* | is the yield stress of the wall in the ith course; |
| *Jef* | is the joint efficiency factor (see 7.2.1). |

(6) Alternatively, the following fully defined procedure may be used to obtain the height over which each planned plate thickness *t*i may be used to resist liquid and vapour pressure.

NOTE 1 The following procedure begins with the assumption that the wall will be constructed using pre-defined thicknesses *t*i of plate (as rolled), probably with pre-defined widths of plate (pre-defined strake heights). The procedure has the advantage that the resulting depths *h*i to the base of each course are clearly deduced, giving a smooth connection to the following procedure that assesses the buckling resistance under external pressure and wind.

NOTE 2 The following direct design method replaces the previous 30 cm rule method used in other standards, which was not technically correct.

(7) The design value of the unit weight (specific weight) of the liquid should be found as

 (7.3)

where:

|  |  |
| --- | --- |
| *γ*d | is the design value of the unit weight of the liquid; |
| *γ*FLl | is the partial factor on liquid pressures (see EN 1990:2023, Annex A.4); |
| *ρ* | is the upper characteristic mass density of the stored liquid; |
| *g* | is the acceleration due to gravity; |
| *γ*U | is the upper characteristic value of the unit weight (specific weight) of the stored liquid. |

(8) The relative effect of the vapour pressure above the surface of the liquid *h*V should be found as

 (7.4)

where:

|  |  |
| --- | --- |
| *p*V,Ed | is the design value of the vapour pressure above the liquid surface (i.e. the characteristic value of pressure multiplied by the partial factor *γ*FV for vapour pressure according to EN 1990:2023, Clause A.4; |
| *hV* | is the height of wall required to accommodate the internal vapour pressure. |

(9) The rate of increase of achievable depth with wall thickness *k*yi is given by

 (7.5)

where:

|  |  |
| --- | --- |
| *γ*d | is the design value of the unit weight of the liquid (Formula (7.3)); |
| *γ*M0 | is the partial factor for plastic failure (see Table 4.4); |
| *fyi* | is the yield stress of the wall in the *i*th course; |
| *jef* | is the joint efficiency factor as defined in (5). |

NOTE Formula (7.5) involves quantities that are frequently stated in inconsistent units (e.g. N/mm2, kN/m3 and m). Care is needed to ensure that the value of *k*y is without units to give a height in Formula (7.6) (see 3.4(2)).

(10) For a chosen wall thickness *t*i with yield stress *f*y, the depth *h*i,0 to which this thickness may be used below the wind girder, curb angle or the top of the cylindrical wall where it is attached to a roof, is given by

 (7.6)

where:

|  |  |
| --- | --- |
| *r*T | is the radius of the middle surface of the tank wall; |
| *t*i | is the thickness of the *i*th course; |
| *h*L | is the maximum liquid height (MNOL or MDLL according to whether a normal or an accidental loading condition is being considered) (Figure 3.1); |
| *hMDLL* | maximum design liquid level height above the base (MDLL) (Figure 3.1); |
| *hMNOL* | maximum normal operating liquid level height above the base (MNOL) (Figure 3.1); |
| *h*S | is the height of the cylindrical shell from the base to the top of the cylindrical wall, the curb angle or the primary wind girder, whichever is less (Figure 3.1); |
| *hF* | is the height of the freeboard above the maximum liquid height (Figure 3.1); |
| *hV* | is the additional height of wall required to accommodate the internal vapour pressure (Formula (7.4)). |
| *h*i,max | is the vertical distance from the wind girder, curb angle or the top of the cylindrical wall (as appropriate) to the lowest point that thickness *t*i may be used; |
| *i* | is the course number, measuring from the maximum design liquid surface downward; |
| *f*yi | is the design yield strength of the steel for this plate of thickness *t*i; |

NOTE 1 The reference height *h*i,max found in Formula (7.6) is chosen so that the definitions of height within the shell wall will be appropriate for determining the resistance to buckling under wind, using a consistent notation.

NOTE 2 The following procedure is designed to distinguish clearly between the possible height of a course *h*i,max and the subsequent chosen height *h*i, which is to be used in determining the resistance against wind and external pressure

(11) Using the chosen sheet height of rolled plate, the integer number of strakes that can be placed before the level *h*i,max is reached can be found. The first thickness *t*1, leading to level *h*1,max can then be used to define the actual height of the first course *h*1. The next sheet thickness *t*2 can then be used in Formula (7.6) to find the lowest point *h*2,max below the liquid surface at which it can be used. This process may be repeated until the complete wall has been defined and the course heights *h*1 to *h*n chosen. Where Formula (7.6) gives a depth that exceeds the height of the wall *H*S, no further changes in wall thickness are required.

NOTE The added height  is based on the restraining effect of the thicker plate below the course being treated, which permits the point of plate thickness change to be lower (see Figure 7.1). This was traditionally defined as 300 mm, which is an approximate value that is usually safe but is no longer required.

Key

|  |  |
| --- | --- |
| X | circumferential membrane stress (MPa) |
| Y | axial coordinate (mm) |
| 1 | peak hoop stress |
| 2 | line of joint |
| 3 | stress without thickness change effect |

Figure 7.1 — Circumferential membrane stress variation at a wall thickness change in a stepped wall cylindrical tank under liquid pressure

## Design for resistance to external pressure and wind

### General

(1) The buckling resistance of a stepped wall tank should be assessed using the provisions of EN 1993‑1‑6, but the following simplified version may be used for tanks in Tank Groups 1 and 2.

(2) The procedures defined in 7.4.2, 7.4.3 and 7.4.4, may be used provided that the top of the shell is adequately restrained against out of round displacements by the roof or a primary ring (see EN 1993‑1‑6).

NOTE Here the top of the shell refers to the wind girder, the curb angle or the roof to cylinder junction, as appropriate.

(3) Where the tank has a fixed roof, which can be either an unsupported roof or a roof supported on a roof supporting structure (see 8.2(3)), and the roof or its support structure is structurally connected to the tank wall, the roof may be considered to provide adequate restraint to the wall to anchor the top of a shell wall buckle under external pressure. If the effective section of the natural ring composed of the connected parts meets the requirements of 7.4.2, a primary ring may be omitted.

(4) An open-top tank should be provided with a primary ring which is located at or near the top of the uppermost course that fulfils the requirements given below.

(5) If the lower edge of the shell is effectively anchored to resist vertical displacements against uplift during buckling, the primary ring stiffener may be designed by satisfying both the strength and the stiffness requirements given EN 1993‑1‑6.

(6) If the shell is of uniform thickness, the buckling assessment should be carried out using the provisions of EN 1993‑1‑6. If its lower edge of the shell is not effectively anchored to resist vertical displacements during buckling, the boundary condition of axially free during buckling should be used.

(7) If the shell has varying thickness down its height (stepped wall) and lower edge of the shell is not effectively anchored to resist vertical displacements during buckling, the buckling assessment may be carried out using computational analysis as defined in EN 1993‑1‑6. Alternatively, the provisions of 7.4.3 may be used.

NOTE The distinction made in (5) and (6) refers to a difference in the buckling resistance under external pressure according to whether vertical uplift displacements can occur during buckling or not. Axial restraint at the base of a potential buckle increases the buckling resistance. However, any buckle that does not extend to the base of the tank is adequately axially restrained by the thicker shell courses beneath the base of the buckle [3] and [4], so the requirement in (6) has only limited application.

### The primary ring to provide top boundary for buckling under external pressure and wind

(1) The primary ring, located at the top of the cylindrical shell, should be designed to be adequately stiff to ensure that calculations concerning buckling of the shell wall under external pressure are safe.

(2) The primary ring should also be designed to have adequate bending resistance in relation to the bending moments induced by unsymmetrical external pressures under wind.

(3) The circumferential bending moments that develop in the primary ring may be assessed using computational linear analysis (LA) according to EN 1993‑1‑6. Where this is not used, the hand calculation procedure set out in Annex D of prEN 1993‑1‑6:2023 should be followed.

(4) The circumferential bending resistance of a primary ring may be evaluated using the procedures defined in EN 1993‑1‑1.

(5) The stiffness required of a primary ring to ensure that the buckling resistance of the shell wall is attainable, see Annex D of prEN 1993‑1‑6:2023.

### Stepped shell wall design for buckling under external pressure and wind

(1) If the shell has varying thickness down its height (stepped wall), the buckling assessment may be carried out using computational bifurcation analysis (LBA) as defined in EN 1993‑1‑6. Alternatively, the provisions of this subclause may be used.

NOTE Where the value of *R*cr derived from an LBA analysis is greater than 2,0, a more complete analysis is typically not necessary.

(2) Since buckling under external pressure reduces as the height of the potential buckle increases, larger potential buckles have a lower resistance and are more critical. But where thicker lower courses play a part in the potential buckle, they lead to higher resistances. Thus the most critical height of the buckle should be found by examining different possible buckle heights.

Key

|  |  |
| --- | --- |
| 1 | mth course |

Figure 7.2 — Stepped wall cylindrical tank notation

(3) Each potential buckle should be assumed to be anchored at the top by the primary ring, and the base of the buckle should be taken at a change of wall plate thickness. Each potential buckle height is denoted by the notation *h*bm, where *m* indicates that the base of the buckle is at the bottom of the *m*th course (Figure 7.2).

(4) Different potential buckle heights *h*bm should be examined until it is found that the stable height of shell *H*b (Formula (7.15)) is less than the height *h*bm. The provisions of 7.4.4 should then be used to proportion one or more secondary rings.

(5) For each course within the buckle, the value of the transformed course height *h*Ti is found as

 (7.7)

(6) The equivalent thickness *t*eq,m of the shell over the physical height *h*bm of each potential buckle in the top unstiffened part of the cylindrical shell should be found as:

 (7.8)

with *h*T0 = 0 when *i* = 1.

where:

|  |  |
| --- | --- |
| *hi* | is the depth from the curb angle or the primary wind girder to the base of the *i*th course (Figure 7.2); |
| *ti* | is the thickness of each course in turn; |
| *h*Ti | is the transformed height of the *i*th course; |
| *m* | is the number of courses being considered in the potential buckle. |

NOTE Where there is only one course in the tank, *t*eq,m is the same as the uniform thickness *t*1 and *hbm* is the full height of the tank.

|  |  |
| --- | --- |
|  |  |
| **a) wind pressure distribution around shell circumference** | **b) equivalent axisymmetric pressure distribution** |

Key

|  |  |
| --- | --- |
| 1 | wind |
| 2 | stagnation point |

Figure 7.3 — Transformation of typical wind external pressure load distribution

(7) The wind pressures *q*w,Ed(θ) should be taken as the values at the top of the cylindrical wall.

NOTE Inward pressures *q* are here treated as positive (outward pressures are denoted as *p*).

(8) The non-uniform wind pressure distribution *q*w,Ed(θ) on the cylinder (see Figure 7.3) may, for the purpose of tank buckling design, be substituted by an equivalent uniform external pressure *q*eq,Ed given by:

*q*eq,Ed = *k*w *q*w,max,Ed (7.9)

where *q*w,max,Ed is the maximum wind pressure (stagnation pressure), and *k*w should be found as:

 (7.10)

but with the value of *k*w limited to being inside the range 0,65 ≤ *k*w ≤ 1.

NOTE Formula (7.10) provides a simpler treatment for wind buckling than EN 1993‑1‑6, by using the simple treatment in EN 1993‑1‑6, and adopting *C*q = 1,0.

(9) The design pressure *q*Ed should be taken as:

*q*Ed = *q*eq,Ed + *q*s,Ed (7.11)

where *qs*,Ed is the internal suction caused by venting, internal partial vacuum or other phenomena.

(10) The critical buckling pressure *q*Rcr,m of the potential buckle of height *hb*m should be calculated as:

 (7.12)

where *Cb* = 1,15 if the base of the wall is anchored or *h*bm does not extend to the base, but *C*b = 0,92 if the buckle extends to the base and the tank is unanchored.

(11) The design value of the buckling resistance pressure of the imperfect shell should then be found as:

 (7.13)

where:

|  |  |
| --- | --- |
| *αθI* | is the imperfection reduction factor under external pressure; |
| γM1 | is the partial factor for buckling (see Table 4.4) |

(12) The reduction *α*θI due to geometric imperfections is given in Table 7.1, based on the Fabrication Tolerance Quality Class as defined in EN 1993‑1‑6.

NOTE The values given in Table 7.1 provide a convenient conservative approximation to those defined in EN 1993‑1‑6.

Table 7.1 — Values of imperfection reduction factor *α*θI for buckling under external pressure

|  |  |  |
| --- | --- | --- |
| **Fabrication tolerance quality class** | **Description** | *αθ*I |
| Class A | Excellent | 0,75 |
| Class B | High | 0,65 |
| Class C | Normal | 0,50 |

(13) It should be verified that

 (7.14)

(14) As the chosen value of *m* increases, it can be found that *q*Rd,m is less than *q*Ed. When this first occurs, the value of *m* should be noted as *m*lim, indicating that the limiting course at which the unstiffened shell can be stable under the design loads has been passed. It is then necessary to place a secondary ring within the course defined by *m*lim to preserve adequate buckling resistance above the secondary ring.

(15) Where *q*Rd,m is found to be less than *q*Ed, a secondary ring should be designed using the provisions of 7.4.4.

(16) To determine the lowest viable location for the secondary ring, the above calculation for the height *h*bm should be repeated with *h*bm assigned slightly reduced values *h*bm,red larger than *h*m-1 until Formula (7.14) can be satisfied. The resulting reduced value *h*bm,red identifies the lowest possible position at which the secondary ring can be placed which is then termed the limiting value *h*bm,lim = *h*bm,red. The provisions of 7.4.4 should be used to design the secondary ring no lower than the level *h*bm,lim.

(17) In an alternative procedure, the height of a buckle that may be taken to be stable without a secondary ring may be determined as:

 (7.15)

in which , and *ka* is given by:

 (7.16)

 (7.17)

where:

|  |  |
| --- | --- |
| *ka* | is the buckling reduction factor for the effect of axial compression (Formula (7.17)); |
| *q*Ed | is the design pressure on the shell wall (Formula (7.11)); |
| *t*eq,m | is the equivalent thickness in the potential buckle (Formula (7.8)); |
| *f*q |  = 97 for Fabrication Tolerance Quality Class A (see EN 1993‑1‑6); |
| *f*q |  = 56 for Fabrication Tolerance Quality Class B (see EN 1993‑1‑6); |
| *f*q |  = 36 for Fabrication Tolerance Quality Class C (see EN 1993‑1‑6); |
| *σ*x,Ed | is the compressive axial membrane stress at the base of the buckle induced by all loads above the base of the assessed buckle. |

(18) The axial stress *σ*xEd induced in the shell by loads other than the external pressure or wind should be properly evaluated using the relevant load combination.

NOTE 1 The buckling resistance of a cylindrical shell under external pressure or wind is reduced if there is an axial load present at the same location. The data on this phenomenon is all obtained from tests and theory in which a uniform thickness cylindrical shell is subject to a uniform external pressure and a uniform axial compression throughout its height. The factor *ka* provides a crude approximation to this effect.

NOTE 2 The axial compressive membrane stress is derived from all roof loads and any other introduced loads. The overturning moment of the unsymmetrical wind pressure produces some tension on the windward side of the shell, but this effect is not considered in this treatment.

NOTE 3 The equivalent thickness *t*eq.m is used in Formula (7.17) to ensure that the effect of an axial compressive membrane stress is evaluated for a thickness that is representative of the full buckle height since Formula (7.17) provides an estimate of the interaction between axial compression and external pressure over the full height of the buckle.

(19) When *ka* < 1, the procedure set out in (17) is only applicable if both of the following conditions are met:

 (7.18)

 (7.19)

where *hb*m is the height of the potential buckle.

NOTE 1 The restrictions given by Formulae (7.18) and (7.19) ensure that axial compression buckling will be in the elastic range, which is required for Formula (7.17) to be valid.

NOTE 2 Formula (7.17) will only produce a credible value if the compressive axial membrane stress *σ*x,Ed does not exceed the characteristic value of the buckling resistance under pure axial compression of the imperfect cylinder, according to EN 1993‑1‑6.

NOTE 3 The above Formulae (7.15) to (7.17) can sometimes be very conservative (especially in the case of very short courses). The provisions of EN 1993‑1‑6 can always be used to provide a more economic design.

(20) Where the height of a potential buckle *h*bm exceeds the height *H*b (Formula (7.15) that can be taken to be stable without a secondary ring, one or more secondary rings should be placed on the cylinder wall, according to the provisions of 7.4.4.

### Secondary rings to increase the buckling resistance under external pressure and wind

#### First secondary ring

(1) A rational method should be used to determine the required size of one or more intermediate ring stiffeners. The method defined in EN 1993‑1‑6 may be used.

(2) It may also be useful to apply the provisions of prEN 1993‑1‑6:2023, 9.7 using the LBA-MNA method to obtain the elastic critical mode and critical buckling pressure and to estimate the plastic reference pressure for any proposed design including ring stiffeners.

(3) A secondary ring should not be located closer than of a circumferential seam on the cylindrical shell, where *ti* is the thinner of the two courses at that seam.

(4) Where a secondary ring stiffener is required to resist buckling under external pressure or wind, as identified using the procedure of 7.4.3, the dimensioning of secondary ring or rings to prevent external pressure buckling of the shell should be undertaken using the provisions of EN 1993‑1‑6. For tanks in Tank Groups 1 or 2, the following simplified procedure may be used.

(5) The minimum second moment of area of the first secondary ring should be found as:

 (7.20)

*h*r1 = *h*bm,lim (7.21)

and *h*bm,lim was found using the procedure of 7.4.3(16).

Key

|  |  |
| --- | --- |
| 1 | Ring *j* |

Figure 7.4 — Secondary ring locations

#### Additional secondary ring

(1) Following design of the first secondary ring, it is possible that one or more further secondary rings may be required. The procedure defined in 7.4.3 should be used, with the first secondary ring height replacing the position of the wind girder, and the heights of courses below it treated as if they were a complete tank. This permits the next part of the tank to be similarly checked and an additional secondary ring inserted as defined in 7.4.4.1.

(2) The procedure of (1) should be repeated each time a further secondary ring is found to be necessary.

#### Tank design using multiple secondary rings

(1) Where multiple rings of identical size are used, a more detailed assessment according to 7.4.4.1 and 7.4.4.2 may be carried out, or else the second moment of area of each secondary ring should satisfy:

 (7.22)

in which *m*B is the critical circumferential buckling mode (Formula (7.25)), and the circumferential force acting in this part of the wall is given by:

 (7.23)

with

 (7.24)

and *m*B assessed as:

 but  (7.25)

where:

|  |  |
| --- | --- |
| *j* | is the secondary ring number, counting from the primary ring or roof junction; |
| *hS* | is the total height from the primary ring or roof junction to the bottom edge of the cylindrical wall; |
| *rT* | is the radius of the middle surface of the cylindrical shell wall; |
| *q*j,Ed | is the inward design pressure at the secondary ring *j*; |
| *a*j | is the vertical height between secondary ring *j* to the secondary ring (*j*+1) below it, or else where there is no lower secondary ring, to the bottom of the cylindrical wall; |
| *a*j-1 | is the distance from secondary ring *j* to the next secondary ring above it, or where there is no higher secondary ring, to the primary ring or roof to shell junction; |
| *t*j | is the mean value of the shell thickness over the range of *a*j; |
| min(*a*j *t*j) | is the minimum value of *a*j *t*j over the complete wall height *HS*; |
| *m*B | is the expected circumferential buckling mode; |
| *I*θ,j | is the second moment of area for circumferential bending of secondary ring *j;* |
| max *I*θ,j | is the highest value of *I*θ,j among all the secondary rings. |

NOTE The background to Formula (7.25) can be found in [5].

## Differential settlement

### General

(1) Where the cylindrical shell is uniformly supported on a foundation, the consequences of minor differential settlement between different locations around the circumference should be considered.

(2) The foundation should be designed according to the provisions of EN 1997 with attention to the potential for local differential settlements.

NOTE The tolerance requirements defined in EN 14015 are usually sufficient for ambient temperature tanks.

(3) The design documentation should specify the limitations on differential settlement to be achieved by the foundation.

NOTE The specification of the limitations on differential settlement to be achieved by the foundation is an essential part of the design documentation for a tank.

(4) The three components of differential settlement should be treated separately:

a) uniform settlement (which may affect connected structures and piping);

b) tilt settlement (which may lead to ovalisation due to different pressures at a single axial coordinate in a tank);

c) local differential settlements (which can lead to serious radial distortions of the cylindrical shell wall that are especially important in tanks with floating roofs).

### Local differential settlement in tanks with floating roofs

(1) Where the tank has a floating roof, very minor differential settlements that occur over a short distance around the circumference can have a major impact on the radial deformations of the cylindrical shell wall and should be limited to the interface flatness tolerances defined in EN 1993‑1‑6.

(2) The analysis of local differential settlements should consider both the axial membrane stresses induced by local deviations of support, and the local radial deformations that arise from local differential settlement.

(3) To evaluate the acceptability of the predicted deformations, they should be added to the maximum radial deviation tolerance (out of roundness) defined in EN 1993‑1‑6 at the same level. Where the tank has a floating roof, the resulting summation should be assessed against the limits of acceptable displacement of the roof seals.

(4) The method and criteria to determine the buckling resistance of a cylindrical shell under local elevated axial stresses caused by local differential settlement are given in EN 1993‑4‑1.

(5) Differential settlement occurring during construction can lead to severe distortions of a cylindrical shell wall that are not easily remedied. Care should be taken to ensure that the foundation is securely stable before construction of the tank begins.

## Support arrangements for a cylindrical shell

(1) Where a cylindrical shell is uniformly supported on a foundation, the provisions of Clause 10 should be met.

(2) Where an elevated cylindrical shell is supported by a skirt, the design should satisfy the provisions of EN 1993‑4‑1.

(3) Where an elevated cylindrical shell is supported with columns that are attached to the exterior surface of the shell over a limited height (known as engaged columns), the design should satisfy the provisions of EN 1993‑4‑1.

(4) Where a cylindrical shell is discretely supported from beneath on columns, local supports or other devices, the provisions of EN 1993‑4‑1 for this condition should be satisfied.

(5) Local supports beneath the wall of a cylinder should satisfy the provisions of EN 1993‑4‑1.

(6) Tanks discretely supported with columns beneath a hopper should satisfy the provisions of EN 1993‑4‑1.

(7) Local ribs for load introduction into a cylindrical shell wall should satisfy the provisions of EN 1993‑4‑1.

# Design of circular roof structures

## General

(1) Steel tank roofs should be so proportioned that the basic design requirements for the ultimate limit states given in Clause 4 are satisfied.

(2) The design of the roof structure should include all load cases including:

— internal and/or external pressure on the roof;

— dead and live loads of the roof plating and structure;

— snow and ice loads;

— wind loads that can be unsymmetrical;

— local horizontal and vertical loads applied to nozzles, attachments and structures supported by the roof;

— seismic loads (see 1.1(7)).

(3) The safety assessment of a spherical or conical shell should be carried out using the provisions of EN 1993‑1‑6, except where this standard provides alternative provisions.

(4) The safety assessment of a flat or inverted cone roof should be carried out using the provisions of this standard using a roof supporting structure.

(5) The safety assessment of the roof supporting structure should be carried out using the provisions of EN 1993‑1‑1.

(6) The design of a floating roof or floating cover should be performed using the provisions of EN 14015 where they are not contradictory to the Eurocodes and EN 1090‑2.

(7) The roof should be checked for:

— resistance to buckling;

— resistance of the joints (connections);

— resistance to rupture under internal or external pressure.

(8) The lack of symmetry of wind loading on a roof may be represented by an eccentricity of the total roof external pressure loading at 0,1 *d,* where *d* is the diameter of the cylindrical wall of the tank.

(9) The roof plating should satisfy the provisions of EN 1993‑1‑6, but the provisions set out in 8.3 to 8.5 may be used instead.

NOTE Clauses 8.3 to 8.5 contain many special provisions applicable only to tanks that are more detailed and practical than the general rules of EN 1993‑1‑6.

## Alternative roof structural forms

(1) A roof may have a spherical, conical, torispherical or toriconical shape.

(2) A roof may alternatively be flat or in the form of an inverted cone.

(3) Where high internal pressures occur above the liquid surface, a torispherical or toriconical shape should be considered.

(4) A metal roof structure in one of the shapes described in (1) can either be unsupported or supported by structural members. A flat or inverted cone roof should be supported by structural members. These structural members are termed a roof supporting structure.

NOTE 1 The term “unsupported” for a roof is used to indicate that the roof is supported only by the cylindrical shell beneath its outer edge.

NOTE 2 The roof supporting structure refers to structural members or a structural framework that provides direct support to the roof plating or sheeting.

(5) The roof supporting structure is here assumed to be a system of radially oriented rafter structural members, which commonly have circumferentially oriented purlins to support the roof plating directly (see 3.1.27 and 29). In smaller tanks, the purlins may be omitted.

(6) The roof can alternatively consist of a membrane, which can be either single skinned or double skinned. Where this roof form is used, internal pressure is normally used to maintain the shape, and appropriate provisions should be made to ensure that the required pressure is maintained under all design situations.

## Considerations for individual structural forms

### Unsupported shell roof structure

(1) Unsupported shell roofs should be of butt-welded or double-welded lap construction.

(2) The roof should be designed according to the provisions of EN 1993‑1‑6.

(3) The design of the roof should take account of the stiffness of the primary ring or girder in resisting radial displacements and providing structural support to its edge boundary.

(4) Where double-welded lap joints are used in the plating, the joint eccentricities cause a reduction of resistance against both buckling and the plastic limit state. These reductions should be taken into account in the model for the analysis in accordance with the provisions of EN 1993‑1‑6.

NOTE Where a torispherical or toriconical roof is used, it is possible for buckling to occur in the knuckle region under internal pressure (see [6]).

## Conical, spherical dome or flat roof with rafter or truss supporting structure

### Plate design general

(1) The roof plating in a conical or spherical dome roof may be designed using large deflection theory (see 8.4.2).

(2) The metal roof plating may be:

a) supported by the roof supporting structure with limited connections between the plating and the structure;

b) attached to the roof supporting structure.

(3) Where the roof supporting structure is external, the form defined in (2)b) should be used.

(4) Where frangibility of the roof is required, the form defined in (2)a) should be used.

NOTE A limited amount of attachment is useful, as indicated in (5).

(5) Where roof frangibility is required, roof plates may only be attached to the internal roof supporting structure subject to the following restrictions:

a) The roof plates should not be attached to the supporting structure within 2 m of the eaves;

b) Unless a more precise verification of the safe venting of the roof is undertaken, in the zone adjacent to the eaves, at least 12 radial tear seams should be arranged between adjacent roof plates.

(6) If a single fillet weld is arranged with a lap joint, special considerations should be given to the corrosion protection in the joints.

(7) Where compressive stresses can develop in the roof plating, the resistance to buckling should be checked (see EN 1993‑1‑6).

### Conical roof plate design using nonlinear theory

(1) The roof plating in a conical roof may be designed using the following procedure based on an approximate large deflection theory treatment. The procedure assumes that the roof plate is in a post-buckled state under a net downward pressure due to snow, ice, wind and superposed loads on the plate alone of *q*p,Ed. The plate is treated locally as a membrane of cylindrical shape between the rafters or purlins, spanning over the local distance *b*r between the rafters or purlins at the chosen location in the roof.

NOTE 1 Where the plating is supported directly on rafters, this procedure uses a location where the rafter separation is large.

NOTE 2 The pressure *q*p,Ed differs from the total vertical pressure load *q*v,Ed relevant to the rafter and primary ring design because only the external loading and the weight of the plate are involved. The total vertical pressure load *q*v,Ed (see 8.4.4(5) and 8.4.5(2)) includes the weight of the supporting structure.

(2) Initially, the plate of thickness *t*pl is assumed to sag by the deformation Δest which may be taken as

 (8.1)

(3) The local radius of curvature of the deformed dished membrane *r*geo is given by

 (8.2)

(4) The angle subtended by the cylindrical sector of the membrane between the rafters is found as

 (8.3)

(5) The plating is assumed to deform into a simple cylindrical surface which supports the downward pressure by membrane action. The full arc length of the membrane *l*geo is given by

 (8.4)

(6) With a downward pressure *q*p,Ed normal to the roof plate alone, the design local circumferential membrane stress *σ*θd is given by

 (8.5)

where *r* is the local thickness of the roof plate.

(7) The circumferential membrane strain *ε*θd may be simply taken as

 (8.6)

(8) The deduced deformed length *l*def of the membrane is found from the strain εθd as

 (8.7)

(9) The above calculation should be repeated with different values Δest and iteratively adjusted until *l*geo ≈ *l*def.

(10) The final value of the circumferential membrane stress *σ*θd should then be verified to satisfy

 (8.8)

### Design of the roof supporting structure

(1) The design of the roof supporting structural members should satisfy the requirements of EN 1993‑1‑1.

(2) The global stability of the roof supporting structure should be verified.

(3) A roof supporting frame structure should either be braced or structurally connected to the roof plating.

(4) A metal roof supporting structure may be placed below the roof plating or above the roof plating.

(5) If adjacent parts (e.g. polygonal rings) of the roof supporting structure are intended to be used with a clevis bearing for the rafter, the joint design should provide sufficient lateral stiffness to prevent instability.

(6) If the roof plating is attached to the roof supporting structure an effective width of this plating may be taken as part of the supporting structure. This total effective width may be taken as 15ε*t* unless a larger value is confirmed by an analysis.

(7) With column supported roofs, special consideration should be given to the possibility of settlement of the foundations.

(8) The following aspects should be taken into account in the supporting structure design.

a) Treatment of welds in rafters;

b) Stability of external stiffeners welded to the roof;

c) The additional forces, moments and torques in rafters where the connections are eccentric.

### Flat or inverted cone roof design

(1) For flat or inverted cone roofs, the plating should be supported on a roof supporting structure.

(2) The roof supporting structure may be arranged below the roof plating or above the roof plating.

(3) The roof supporting structure according to 8.4.2 may be supported by columns.

(4) The metal roof plating may be:

a) supported by the roof supporting structure without connections between the plating and the structure;

b) attached to the roof supporting structure.

(5) Under the action of distributed loads arising from imposed load, snow load, wind load, permanent load and pressure, the maximum vertical component should be taken as the design value *q*v,Ed acting downwards, with *q*v,Ed taken as negative if it acts upwards.

(6) The roof supporting structure should meet the requirements of EN 1993‑1‑1.

(7) Where the roof has the form of an inverted cone, the tensile forces developing in the rafters should be accounted for in the rafter design. The horizontal component of a tensile forces at the attachment to the primary ring should be treated in the same way as horizontal components in shell roofs (see 8.5) and the ring treated as in Clause 9.

NOTE The treatment of forces from inverted cone roofs can be treated as described for conical hoppers in EN 1993‑4‑1.

### Dome roof design

|  |  |
| --- | --- |
|  |  |

Key

|  |  |
| --- | --- |
| 1 | roof profile |
| 2 | tank axis |

Figure 8.1 — Notation for spherical dome and conical roof

(1) Spherical dome roofs (Figure 8.1) supported on rafters under the action of distributed loads arising from imposed load, snow load, wind load, permanent load and pressure, the maximum vertical component should be taken as the design value *q*v,Ed acting downwards, with *q*v,Ed taken as negative if it acts upwards.

(2) The total design vertical force *P*Ed on each rafter should be taken as:

 (8.9)

where:

|  |  |
| --- | --- |
| *β*r | is the half angle of the horizontal angular separation of two rafters (= π/*n* radians) |
| *N* | is the number of rafters; |
| *r*T | is the radius of the middle surface of the tank vertical wall; |
| *q*v,Ed | is the maximum vertical component of the design distributed load including the dead weight of the supporting structure (downward positive); |
| *P*Ed | is the total design vertical force per rafter. |

(3) The bracing between rafters should be designed to provide a stabilizing force greater than 1 % of the sum of the axial forces in the stabilized members.

(4) The normal force *N*Ed in each rafter for cross-section resistance design according to EN 1993‑1‑1 may be obtained from:

 (8.10)

where:

|  |  |
| --- | --- |
| *hS* | is the rise of the tank roof, see Figure 8.1; |
| Gri.Ed | is the design self weight per unit length of a rafter; |
| eN | is the eccentricity factor for the normal force connection to the primary ring. |

with:

|  |  |
| --- | --- |
| eN |  = 1,0 for a concentric force connection; |
| eN |  = 1,1 for an eccentric force connection. |

(5) The maximum bending moment *M*Ed in each rafter for cross-section resistance design according to EN 1993‑1‑1 may be obtained from:

 (8.11)

 (8.12)

where:

|  |  |
| --- | --- |
| eB | is the eccentricity factor for the bending moment connection to the primary ring. |

with:

|  |  |
| --- | --- |
| eB |  = 1,0 for a concentric connection; |
| eB |  = 1,2 for an eccentric connection. |

NOTE For dome roofs, the above formulae omit the effects of arch action in the rafters. This effect is not specifically addressed in EN 1993‑1‑1.

(6) Formulae (8.10) and (8.11) are valid provided that the following conditions are met:

 (8.13)

 (8.14)

 (8.15)

 (8.16)

where:

|  |  |
| --- | --- |
| *hS* | is the rise of the tank roof, see Figure 8.1; |
| *rT* | is the radius of the cylindrical wall of the tank, see Figure 8.1; |
| *xS* | is the vertical height of a point on the roof at coordinate *r*S, see Figure 8.1; |
| *b*k | is the flange width of the centre ring, see Figure 8.2; |
| *h*k | is the vertical distance between the flanges of the centre ring, see Figure 8.2; |
| *A*1 | is the area of the top flange of the centre ring, see Figure 8.2; |
| *A*2 | is the area of the bottom flange of the centre ring, see Figure 8.2; |
| *I*y | is the second moment of area of the rafter about its horizontal axis. |

(7) The effective width of the shell plate may be included in assessing the effective cross-section of the rafter only if these two parts are structurally connected. Where intermittent welds or other connectors are used, they shall satisfy the requirements of EN 1993‑1‑8.

(8) If the second moment of area of the rafter *I*y varies along the length of the rafter (e.g. due to the variable effective width of roof plates when they are connected to the rafters) the value of *I*y at a distance 0,5*r* from the tank axis may be used in the design of the centre ring.

### Roof centre ring

(1) Provided that the conditions given in 8.4.3 or 8.4.5, as appropriate, are satisfied, the design of the centre ring may be verified by checking only its lower chord according to (2).

Key

|  |  |
| --- | --- |
| 1 | profile section separating flanges |
| 2 | neutral axis of rafter |
| 3 | tank axis |
| 4 | neutral axis of A1 and A2 for bending in the plane of the plates |

Figure 8.2 — Roof centre ring

NOTE It is useful to adopt the dimensional restrictions ,  and where *I*y is the second moment of area of the ring about its vertical neutral axis.

(2) Provided that there are at least 10 uniformly spaced rafters, the design value of the member force *Nr*,Ed and bending moment *M*r,Ed for the central ring may be calculated using:

 (8.17)

 (8.18)

in which:

 (8.19)

where:

|  |  |
| --- | --- |
| *β*r | is the half angle of the horizontal angular separation of two rafters (= π/*n* radians) |
| *N*2,Ed | is the design value of the force in the lower chord of the centre ring; |
| *N*Ed | is the design value of the force in the rafter; |
| *M*Ed | is the design value of the bending moment in the rafter at its inner end; |
| *e*o | is the vertical eccentricity of the rafter neutral axis from the top flange of the centre ring (Figure 8.2); |
| *hk* | is the vertical separation of the flanges of the centre ring (Figure 8.2) |
| *r*k | is the radius of the neutral axis of the centre ring (Figure 8.2). |

### Column supported roof structure

(1) The recommended minimum nominal thickness of roof plating is indicated in Table 8.1.

Table 8.1 (NDP) — Recommended minimum nominal thickness for roof plates

|  |  |
| --- | --- |
| **Material** | **Roof plate thickness** |
| Carbon steels | 5 mm |
| Stainless steels | 3 mm |

NOTE The minimum nominal roof plate thickness for welded tanks can be defined in the National Annex.

(2) The roof plates may be designed using large deflection theory.

(3) The design of the roof supporting structure should satisfy the requirements of EN 1993‑1‑1.

### Bracing and rings where roof plates are not connected to the rafters

(1) In tanks where the roof plates are not structurally connected to the rafters, bracing should be used.

(2) Where the roof exceeds 15 m diameter, the following bracing arrangement should be taken as the minimum requirement. Pairs of adjacent rafters should be selected to be braced together using truss members. These pairs of rafters, termed braced bays, should be placed at even spacing around the tank circumference. A minimum of two braced bays should be used (on a diameter), but the number chosen should take consideration of the tank diameter.

(3) Where a braced roof has a diameter between 15 m and 25 m, an additional circumferential ring should be provided. Where a braced roof has a diameter over 25 m, two additional circumferential rings should be provided.

(4) The bracing between rafters should be designed to provide a stabilizing force greater than 1 % of the sum of the axial forces in the stabilized members.

## Spherical or conical shell roof without roof supporting structure

(1) The following rules may be used for the design of unstiffened shell roofs with butt welded or with single or double lap welded joints.

(2) This procedure for spherical dome roofs may be used if the diameter of the tank is less than 60 m and the distributed load does not deviate significantly from symmetry about the tank axis.

(3) Provided that the maximum local value of the distributed design load is used in (3) to represent the distributed pressure on the roof, possible non-uniformity of the distributed load need not be considered.

NOTE A fixed shell roof is one that is attached to the top of the cylindrical wall. It contrasts with a floating roof on the surface of the stored liquid.

(4) Where a concentrated load is applied, a separate assessment should be made using the provisions of EN 1993‑1‑6.

NOTE This separate assessment is always relevant when platforms or runways are mounted on a tank roof.

(5) The resistance of the roof under the design internal pressure *p*0,Ed should be verified using:

 for a spherical roof (8.20)

 for a conical roof (8.21)

in which:

 for a conical roof (8.22)

where:

|  |  |
| --- | --- |
| *jef* | is the joint efficiency factor; |
| *p*0,Ed | is the outward component of the uniformly distributed design load on the roof; |
| *rT* | is the radius of the middle surface of the cylindrical shell wall; |
| *r*CO | is the radius of curvature for a conical roof; |
| *r*S | is the radius of curvature of a spherical roof; |
| *tCO* | is the conical roof plate thickness; |
| *tS* | is the spherical roof plate thickness; |
| *α* | is the slope of a conical or spherical roof to the horizontal at the eaves (see Figure 8.1) |
| *γ*M0 | is the partial factor for plastic failure (see Table 4.4). |

NOTE The notation for the radius of curvature of conical *r*CO and spherical *r*S shells has been made consistent with that of EN 1993‑1‑6 to avoid the use of *R* which is reserved for resistance.

(6) The joint efficiency factor *jef* should be as defined in prEN 1993‑1‑6:2023, 7.2.2(8).

(7) The stability of a spherical roof under the design external pressure *q*i,Ed should be assessed using the provisions of EN 1993‑1‑6.

(8) A simpler conservative treatment may be deemed to meet the requirements of EN 1993‑1‑6 as:

 (8.23)

where:

|  |  |
| --- | --- |
| *α*s |  = 0,05 for butt welded roofs; |
| *α*s |  = 0,04 for lap welded roofs; |
| *α*s | is the imperfection reduction factor; |
| *q*i,Ed | is the inward pressure component of the uniformly distributed design load on the roof. |

(9) The stability of a conical roof under the design external pressure *p*i,Ed should be verified according to the provisions of EN 1993‑4‑1 or EN 1993‑1‑6.

# Roof to shell junction and primary ring (eaves junction)

## General: conventional arrangement

(1) The forces that develop in conventional roofs with the apex above the level of the ring are treated in 9.1 to 9.3. Where an inverted cone roof is involved, see 9.4.

(2) The roof to cylinder junction and primary ring (eaves junction) should be designed to resist the circumferential tensile force induced by the total downward vertical load from the roof (dead weight, snow, live load and internal negative pressure).

(3) The roof to cylinder junction should also be designed to resist the compressive force induced by net upward loads on the roof (wind and internal positive pressure).

Where a membrane roof is used, the internal pressure required to maintain its form should be recognized as the principal source of upward load on the roof.

NOTE Where a flat roof is used, the following treatment of the edge ring is invalid. A simple beam bending treatment of the primary ring can be used to address the discrete forces from rafters.

(4) The roof to cylinder junction and primary ring should satisfy the requirements of EN 1993‑1‑6 or EN 1993‑4‑1 according to the geometry of the roof. The simplified design method given in 9.2 may be deemed to meet those requirements if the conditions set out here are met.

## Primary ring or girder at the shell to roof junction (eaves ring)

(1) The design of the primary ring should consider the two load cases:

a) Load Case A: the maximum vertical downward design distributed load including the weight of the plating, supporting structure and external vertical loads;

b) Load Case B: the minimum (guaranteed) vertical downward design distributed load from the weight of the plating and supporting structure in the presence of wind and internal vapour pressure (potentially a negative downward load).

(2) The effective section of the primary ring or girder should be evaluated using the provisions of EN 1993‑4‑1 for a transition junction.

(3) However, the simplified treatment shown in Figure 9.1 may be deemed to provide a safe approximation to the precise value of the total effective cross-sectional area *A*et using following procedure:

(4) The reference effective length of the roof plate (conical or spherical) is given by

 (9.1)

(5) The effective lengths of the three contributing plates are given by

 (9.2)

if  then  (9.3)

If , then  (9.4)

(6) The total effective cross-sectional area is given by

 (9.5)

where:

|  |  |
| --- | --- |
| *A*s | is the cross-sectional area of a structural member present; |
| *tr* | is the plate thickness of the roof (either conical or spherical dome); |
| *t1* | is the plate thickness in the top course of the vertical wall. |

(7) Where a structural member is present. (e.g. angle section) (Figure 9.1b), it should not have a horizontal leg outstand length greater than 16*t*a where *t*a is the thickness of the leg outstand.

|  |  |
| --- | --- |
|  |  |
| **a) no structural ring** | **b) with angle section ring** |

Figure 9.1 — Approximate effective section of the primary ring at the shell to roof junction

(8) Where a structural member is included as part of the ring, the centroid of the effective section should be located close to the intersection of the roof and cylindrical shell centroids.

NOTE The circumferential stresses in a structural member that is part of the ring are only fully developed if its centroid is placed close to the intersection of the two plate centrelines, since that is the location where horizontal forces are resisted by circumferential forces. Since this ring is at the top of a thin shell, it can only be effective if it is located within about ℓc/4 of the intersection.

(9) The primary ring effective cross-sectional area *Aet* should be chosen to satisfy both Formulae (9.5) and (9.6):

 (9.5)

 (9.6)

where:

|  |  |
| --- | --- |
| *FA* | in Load Case A, is the design value of the vertical downward force transferred to the cylindrical shell wall from the roof supporting structure, roof plating, and framing supported by the cylindrical shell wall, as well as any external pressure applied to the roof; |
| *FB* | in Load Case B, is the maximum uplift (or minimum guaranteed vertical downward design vertical force) from wind and internal vapour pressure, allowing for the weight of the plating and supporting structure; |
| *α* | is the angle between the roof and a horizontal plane at the roof to cylinder junction. |

(10) Where necessary, an increased plate thickness in the roof or cylindrical wall, or an added structural member may be used to increase the effective cross-sectional area of the ring, treated according to the provisions of EN 1993‑4‑1. The net compression area should be arranged to ensure that its centroid is no further vertically than  from the intersection of the roof and cylindrical shell middle surfaces, where *t*1 is the local thickness of the cylindrical shell.

NOTE For the effect of eccentricity at this joint, see [7].

(11) The maximum tensile force (Load Case A) in the effective area of the primary ring or girder that is connected to the roof should be verified using:

 (9.7)

in which:

 (9.8)

where:

|  |  |
| --- | --- |
| *A*et | is the total effective cross-sectional area of the primary ring indicated in Figure 9.1; |
| *α* | is the slope of the roof to the horizontal at the junction; |
| *q*v,Ed | is the maximum vertical downward component of the design distributed load including the weight of the plating, supporting structure and external vertical loads (all downward positive). |

(12) Where the roof is subject to internal pressure *p*Ed, the potential compressive force (negative) that can develop in the effective primary ring (Load Case B) should be found using:

 (9.9)

where *q*v,min,Ed is the minimum (guaranteed) vertical downward component of the design distributed load from the weight of the plating and supporting structure.

(13) Where *N*Ed in (3) is found to be negative (i.e. compressive), the buckling resistance of the ring should be assessed using the provisions of EN 1993‑4‑1 for the buckling of transition rings. Since the loading comes principally from rafters that restrain in-plane buckling, and the ring is structurally connected to the shell, the only buckling checks required are those for out of plane (torsional and distortional) buckling. The provisions of EN 1993‑4‑1 for this buckling mode are then relevant.

(14) Where the net design distributed load *q*v,Ed is negative (i.e. acts upwards as from internal pressure), the bending moments in the primary ring may be ignored.

(15) Where the separation between adjacent rafters at their points of connection to the primary ring does not exceed 3,25 m, and the design distributed load *q*v,Ed acts downwards, the bending moments in the primary ring may be ignored.

NOTE The rule permitting the bending moments to be ignored is based on the following understanding. Upwards directed loads are transferred to the ring by the roof plates, so a relatively uniform circumferential stress develops in the ring. By contrast, downwards loads are normally carried by the rafters that introduce point loads on the ring. But this concept is only valid for rafters that are directly connected to the ring and roof plates that are not welded to the rafters. This treatment is invalid for outside rafters that end near the eaves junction.

(16) Where the separation between adjacent rafters at their points of connection to the primary ring exceeds 3,25 m, the circumferential bending moments in the primary ring (about its vertical axis) should be taken into account in addition to the normal force in the ring *N*Ed. The bending moments in the ring (positive values inducing tensile stresses on the outside of the ring) should be evaluated using Formulae (9.10) and (9.11).

At the connection between the ring and the rafter:

 (9.10)

At half span between the rafters:

 (9.11)

where:

|  |  |
| --- | --- |
| *β*r | is the half angle of the horizontal angular separation of two rafters (= π/*n* radians). |

NOTE If *q*v,Ed acts in the upward direction, it is taken as negative, causing a change of sign in all the normal forces and bending moments. Note that *q* is defined as an external pressure load on the tank (Figure 3.1b).

## Inverted cone roof primary ring or girder at the shell to roof junction

(1) The provisions of 9.1 and 9.2 may be applied to a roof in the form of an inverted cone, but care should be used in recognizing that the forces in the ring are reversed.

(2) The roof to cylinder junction and primary ring (eaves junction) should be designed to resist the circumferential compressive force induced by the total downward vertical load from the roof (dead weight, snow, live load and internal negative pressure).

(3) The roof to cylinder junction should also be designed to resist the tensile force induced by a net upward load on the roof (internal positive pressure).

(4) The tensile force in the ring should be evaluated using Formula (9.9) and the compressive force using Formula (9.8).

(5) The cross-sectional area *A*et defined by Formula (9.4) should be verified as sufficient to satisfy the tensile force requirement of Formula (9.7).

(6) The cross-section of the effective ring should meet the requirements of 9.2(9) to resist buckling of the ring the calculated compression force.

(7) The bending moments in the ring defined by Formulae (9.10) and (9.11) may be used without modification.

# Tank bottoms and annular plates

## General

(1) The design of the bottom plate (sketch plate) and annular plate of a ground-supported tank should take corrosion into account (see 4.11 (8)).

(2) Special requirements for corrosion loss may be omitted if at least one of the following requirements are fulfilled:

a) Corrosion is prevented for physical reasons (e.g. material selection such as stainless steel or absence of liquids or gases involved in corrosive reactions, such as oxygen/air);

b) Appropriate corrosion protection systems are in place.

(3) Openings in the tank bottom plate are not allowed unless the structural safety is demonstrated by an appropriate structural calculation.

(4) The geometry of the base of the cylindrical shell, annular plate and bottom plate is shown in Figure 10.1.

Key

|  |  |
| --- | --- |
| 1 | tank shell |
| 2 | annular plate |
| 3 | bottom plate |
| 4 | vertical weld in shell wall |
| 5 | full penetration butt weld in annular plate |

Figure 10.1 — Dimensions of cylindrical shell base, annular plate and sketch plate

(5) The annular plate and bottom (sketch) plate should satisfy the strength and toughness requirements of the lowest shell course.

## Annular plates

(1) It is recommended that the annular plate has a minimum nominal thickness *t*ap,mn, excluding any corrosion allowance (NDP) of:

 (10.1)

NOTE 1 The National Annex can define the minimum nominal thickness *t*ap,min.

NOTE 2 The dimensions of the annular plate can be governed by action combinations involving lateral forces (e.g. wind, seismic action) – especially for unanchored tanks for which uplift can be possible.

NOTE 3 The recommended minimum thickness of annular plate is chosen to accommodate the base rotation of the cylindrical wall by a plastic hinge in the annular plate, to avoid alternating plasticity in the weld detail at the bottom of the cylindrical wall. However, it should be noted that this minimum plate thickness can sometimes lead to uplift of the outer edge of the annular plate, potentially leading to corrosion beneath the plate.

(2) Where significant axial forces can develop in the cylindrical shell (e.g. under gravity, wind or seismic action), the annular plate should be designed to distribute the axial force into the foundation.

(3) When considering hydrostatic loading conditions, the radial width of the annular plate inside the cylindrical tank shell *w*ap,I should be larger than *w*ap,I,min:

 but not less than 500 mm if the tank radius exceeds 8m (10.2)

where:

|  |  |
| --- | --- |
| *hL* | is the liquid level above the base in the considered design situation (Figure 3.1); |
| *wap,I,min* | is the minimum exposed inside width (distance from the inner edge of the annular base plate to the inner edge of the shell plate); |
| *tap* | is the thickness of the annular plate, after accounting for the corrosion allowance; |
| *ρ* | is the lowest density of the contained liquid. |
| *g* | is the acceleration due to gravity. |

(4) Where uplift of the shell can occur (e.g. wind or seismic actions), the radial width inside the shell *w*ap,i,lim should be increased to ensure that the inside edge of the annular plate remains in contact with the foundation or supporting structure under the most unfavourable load combination.

(5) The annular plate should extend from the outer surface of the cylindrical shell plate by a distance greater than *w*ap,O,min:

 (10.3)

NOTE In smaller tanks where the sketch plate serves the role of the annular plate, this distance can be reduced.

(6) The radial welds connecting annular plate segments to each other should be full penetration butt-welded (see Figure 10.1). For welding details and verification, see EN 1993‑1‑8.

(7) The attachment of the lowest course of the shell plate to the annular plate should be continuous fillet welds on both sides of the shell plate or a full penetration butt weld. Where an annular plate is omitted, the same rule should apply to the bottom central or sketch plate.

(8) The radial welds connecting annular plate segments to each other should be full penetration butt-welded. For welding details, see EN 1993‑1‑8.

(9) The throat thickness of each fillet weld should be according to the requirements of EN 1993‑1‑8. It should normally be greater than or equal to the thinner of the thickness of the annular plate, or where appropriate, the sketch plate.

NOTE The maximum throat thickness can be defined in the National Annex.

## Bottom central plates (sketch plates)

(1) Bottom plates should be lap-welded or butt-welded. For welding details see EN 1993‑1‑8. The overlap length of the lap joint between the bottom plate and the annular plate should not be less than *l*AP,BP = 50mm. Lap joints between plates of the same thickness should have an overlap length not less than *l*bP,BP = 5 *t*BP.

(2) It is recommended that the nominal thickness *t*BP of the bottom plate in its corroded condition should be at least the values specified in Table 10.2.

Table 10.2 (NDP) — Recommended minimum bottom plate thickness *t*BP for welded tanks after the corrosion allowance has been subtracted

|  |  |  |
| --- | --- | --- |
| **Material** | **Lap-welded bottoms** | **Butt-welded bottoms** |
| Carbon steels | 6 mm | 5 mm |
| Stainless steels | 5 mm | 3 mm |

NOTE 1 The minimum corroded bottom plate thickness *t*BP for welded tanks is given in Table 10.2, unless the National Annex gives different values.

NOTE 2 Further useful information can be found in EN 13445‑3.

(3) If the weight of the tank bottom plates is required to assist in resisting uplift due to internal negative pressure, a larger thickness should be chosen unless a minimum guaranteed residual liquid level (min NOL) is used to assist in resisting this uplift.

(4) Bottom plates supported by parallel girders (elevated bottoms) may be designed as continuous beams according to small deflection theory. If the deformation of the cross section of the supporting girders due to the lateral load is negligible (e.g. concrete beams, hollow sections, beams with heavy flanges), the span of the continuous beam representing the plate may be taken as the distance between adjacent edges of these supporting members, instead of the distance between the centre-lines of the supporting members.

# Openings in the cylindrical shell or roof

## General

(1) Where an opening in the cylindrical shell wall or roof reduces the load carrying capacity or endangers the stability of the structure, the opening should be reinforced.

(2) This reinforcement can be achieved by:

— increasing the thickness of the plate;

— adding a reinforcing plate;

— adding a nozzle body.

NOTE The design against the plastic limit state (LS1) generally governs in the region of high pressure loading (liquid and internal) whereas stability considerations (LS3) are likely to control the design in regions where the plate thickness is small due to low pressures (upper courses).

## Shell nozzles of small size

(1) Nozzles in the cylindrical shell wall with an outside diameter less than 80 mm are classed as small size.

(2) Reinforcement may be omitted, provided that the thickness of the cylindrical shell wall adjacent to the nozzle is not less than *t*T,ref,n given by:

 but not less than *t*nm0 (11.1)

where:

|  |  |
| --- | --- |
| *dn* | is the diameter of the nozzle; |
| *tT,ref,n* | is the thickness of the shell plate near the nozzle; |
| *tnm* | is 2 for carbon steel and 1,2 for stainless steels; |
| *tnm0* | is 5,0 mm for carbon steel and 3,5 mm for stainless steels. |
| *am* | is 14 for carbon steel and 15 for stainless steels. |

(3) The values for *t*T,ref,n given in Table 11.1 may be used in place of Formula (11.1).

Table 11.1 — Minimum nozzle body thickness

|  |  |
| --- | --- |
| **Outside diameter** *d*n **of manhole or nozzle** (mm) | **Minimum nominal thickness** *t*ref,n (mm) |
| **Carbon steel** | **Austenitic and austenitic-ferritic stainless steel** |
| *d*n ≤ 50 | 5,0 | 3,5 |
| 50 < *d*n ≤ 75 | 6,5 | 5,0 |
| 75 < *d*n ≤ 80 | 7,5 | 6,0 |

## Design of cylindrical shell manholes, access doors and nozzles of large size for LS1

(1) Manholes, access doors and nozzles in the cylindrical shell wall with outside diameter greater than 80 mm are classed as of large size. In this subclause these are all referred to as nozzles.

(2) The forces acting on a nozzle from attached piping should be provided to the structural engineer by the piping designer. Possible forces that should be considered are radial, torsional and moments.

NOTE Special provisions in the piping can be used to reduce these forces.

(3) The design may be undertaken using either the area replacement method according to paragraphs (4) and (5) or by the method described in (6) to (8).

(4) A reinforcement of cross-sectional area Δ*A* should be provided in the vertical plane containing the centre of the opening, given by:

Δ*A* = *ks d*o *t*i,ref (11.2)

where:

|  |  |
| --- | --- |
| *ks* |  = 0,75 for welded construction and = 1,0 for bolted construction; |
| *d*o | is the diameter of the hole cut in the shell plate; |
| *t*i,ref | is the thickness required by the design for LS1 for the shell plate without the opening. |

(5) The reinforcing area Δ*A* may be provided by any one or any combination of the following three methods:

a) The provision of a nozzle or a manhole body. The portion of the body which can be considered as reinforcement is that lying within the shell plate thickness and within a distance of four times the body thickness from the shell plate surface unless the body thickness is reduced within this distance, when the limit is the point at which the reduction begins.

b) The addition of a thickened shell insert plate or a reinforcing plate, the limit of reinforcement being such that 1,5*do* < *d*n < 2*do*, where *d*o is the diameter of the opening and *d*n is the effective diameter of reinforcement. A non-circular reinforcing plate may be used provided the minimum requirements are met.

c) The provision of a shell plate thicker than required by the design for LS1 for the shell plate without an opening. The limit of reinforcement is the same as that described in b).

(6) As an alternative to the area replacement method specified in (3) to (5) the reinforcement may be achieved by introducing a barrel-type nozzle reinforcing body that protrudes on both sides of the shell plate by an amount not less than Δ*t*x, given by

 (11.3)

(7) This method should not be used unless the nozzle body is more than 100 mm from the base ring plate or annular plate.

(8) The thickness of the nozzle body should be chosen such that the stress concentration factor *f*s does not exceed 2,0. The stress concentration factor *f*s should be obtained from Formula (11.5) using the replacement factor *f*r. The replacement factor *f*r should be evaluated from:

 (11.4)

and the stress concentration factor *f*s from:

 (11.5)

where:

|  |  |
| --- | --- |
| *ti* | the shell plate thickness required to resist only internal pressure from the stored liquid and overpressure at the nozzle location; |
| *t*n | is the nozzle body total thickness (= *t*T + 2Δ*t*x); |
| *r*n,m | is the mean radius of the nozzle (nozzle middle surface); |
| *r*n,e | is the external radius of the nozzle; |
| *r*n,i | is the inside radius of the nozzle. |

(9) The design of the nozzles of large size may also be calculated according to EN 13445‑3, provided that the appropriate safety requirements of EN 1990 are met.

## Cylindrical shell wall design for LS3 in the presence of shell openings

(1) The potential for buckling of the cylindrical shell under axial compression caused by the increased stresses adjacent to a shell opening should be verified using the computational provisions of EN 1993‑1‑6.

NOTE Useful information on the design of stiffened openings can be found in [8].

(2) The zone around the opening should be reinforced by a cross-sectional area Δ*A* equal to or greater than that removed by the opening. The reinforcement can be provided according to 11.3 or by means of stiffeners in the axial direction.

(3) Where the radius of a circular opening *r*0 is less than one third of the radius *r* of the cylindrical shell, no reduction in the assessed buckling resistance need be made as a result of the opening, provided that the requirement given in (2) is met.

(4) The effect of a circular opening on the stability of a cylindrical shell may be neglected provided that the dimensionless opening size *η* is smaller than *η*max = 0,6, where *η* is given by:

 (11.6)

where:

|  |  |
| --- | --- |
| *rT* | is the radius of the cylindrical shell near the opening; |
| *ti* | is the thickness of the unstiffened shell wall near the opening; |
| *r*o | is the radius of the opening. |

(5) Where the opening is rectangular, the value of *η* may be taken from Formula (11.6) with the equivalent opening radius *r*o taken as:

 (11.7)

where:

|  |  |
| --- | --- |
| *a* | is the horizontal width of the opening; |
| *b* | is the vertical height of the opening. |

(6) If stiffeners in the axial direction are used to reinforce the opening, the cross-section of each stiffener should be reduced towards its ends to prevent the formation of buckles due to stress concentration in the shell plate near the stiffener ends.

## Design of openings in the roof

(1) An opening in the roof should be treated as a large nozzle (see 11.3), except where special provision is made by the use of secondary beams between roof girders.

(2) Where a roof nozzle has diameter below 50 mm and without external loads applied to it, and a minimum wall thickness according Table 11.1, no calculation is required.

# Design for static stability of anchored and unanchored tanks

## Unanchored ground supported tanks

### Uplift

(1) Where a tank is unanchored, the base of the cylindrical wall should be checked to ensure that uplift does not occur under all static design situations (see Figure 12.1b and c).

(2) Where a seismic design is required, the provisions of EN 1998‑4 should be followed.

(3) To ensure that uplift does not occur under extreme wind loading, the minimum normal operating liquid level (min NOL) should be determined and documented.

(4) The component of wind loading (see 7.3) that an overturning moment that induces tension at the base of the wall should be identified with the value of γF chosen for detrimental cases (EN 1990). The destabilizing effect of internal vapour pressure, as indicated in Figure 12.1b, should be included in assessing the wall base tension, using the same value of γF.

(5) The restoring effects of the tank weight and the pressure of liquid in the base should evaluated using the value of γF associated with beneficial loading (EN 1990).

|  |  |  |
| --- | --- | --- |
|  |  |  |
| **a) Anchored tank with guaranteed liquid at all times** | **b) Unanchored tank with uplift forces** | **c) Unanchored tank responding to lateral forces (e.g. wind or seismic)** |

Key

|  |  |
| --- | --- |
| 1 | no pressure |
| 2 | internal vapour pressure |
| 3 | lateral force or pressure |

Figure 12.1 — Anchored and unanchored tanks

### Overturning

(1) The possibility of overturning of a tank should be considered under all static design situations (see Figure 12.1b and c), with special attention to wind loading using the value of γF for wind, coupled with the self-weight of the empty tank using the value of γF associated with beneficial loading (EN 1990).

(2) Overturning under seismic action when the tank is full should be considered using the provisions of EN 1998‑4.

NOTE Information on appropriate calculations to determine the uplift under seismic conditions can be found in EN 1998‑4.

## Anchorage design for anchored ground supported tanks

### General

(1) The anchorage should be attached to the cylindrical shell and not to the annular base ring plate or bottom plate alone.

(2) In determining whether an anchorage is needed, the effect of the internal pressure on the bottom plate may be considered.

NOTE The effect of the internal pressure can only be engaged to give a beneficial effect that reduces anchorage requirements if local uplift of the bottom occurs. This can be undesirable for the long term condition of the tank.

(3) The design should accommodate movements of the tank due to thermal changes and hydrostatic pressure to minimize stresses induced in the shell by these effects.

(4) Where a uniformly supported anchored tank is subject to horizontal loads (e.g. wind) the anchorage forces should be determined according to the provisions of EN 1993‑4‑1.

(5) Where a uniformly supported anchored tank is subject to horizontal loads (e.g. wind) the anchorage forces may alternatively be calculated according to either linear shell bending theory or shell semi-membrane theory. They should not be calculated using beam theory.

NOTE 1 The forces calculated using shell theory are locally much higher than those found using beam theory.

NOTE 2 Design advice pre-dating about 2010 (e.g. [8]) indicated that anchorage forces required to resist wind loads are very much higher than the value derived from global bending. Recent research has shown that the anchorage forces are still higher than the global bending values, but the stiffness of each anchorage detail has a substantial influence on the anchorage force. This updated treatment is now given in EN 1993‑4‑1.

NOTE 3 The updated treatment given in EN 1993‑4‑1 often leads to very conservative assessments of anchorage forces. To obtain less conservative forces, it is necessary to determine the vertical stiffness of the individual anchorage arrangement and to use this in a computational analysis using the provisions of EN 1993‑1‑6 to determine accurate estimates of the real anchorage forces under the given loading condition.

(6) The design of the cylindrical shell to resist the local anchorage forces applied to it by the anchorage should meet the provisions of EN 1993‑4‑1 or EN 1993‑1‑6.

### Anchorage design

(1) Tank anchorage should be provided for fixed roof tanks, if any of the following conditions could possibly cause the cylindrical shell wall and the bottom plate close to it to lift off its foundations:

a) Uplift of an empty tank due to internal design pressure counteracted by the effective corroded weight of roof, shell and permanent attachments (see Figure 12.1b);

b) Uplift due to internal design pressure in combination with wind loading counteracted by the effective corroded weight of roof, shell and permanent attachments plus the effective weight of the product always present in the tank as agreed between the designer, the client and the relevant authority (see Figure 12.1c).

NOTE The effective weight of product that is deemed to be always present in the tank can be significantly less than its true weight due to liquid movement under this extreme design situation.

c) Uplift of an empty tank due to wind loading counteracted by the effective corroded weight of roof, shell and permanent attachments.

(2) In determining the required anchorage forces, the uplift forces due to the wind load should be calculated according to 12.2.1(5). The internal vapour pressure should also be taken into account.

(3) Where it is guaranteed that an appropriate quantity of liquid will always remain in the tank, its weight may be included in the assessment.

(4) If the cylindrical shell is assumed to have a rigid cross-section (beam theory), local uplift can be predicted to occur. Under such conditions, the true flexibility of the whole structure under unsymmetrical loads should be assessed (semi-membrane theory or finite element analysis), and suitable provision should be made for the consequences. In particular, there can be unacceptable distortion of the shell wall far above the point of uplift.

NOTE Where the tank has an internal floating roof, distortion of the cylindrical shell wall due to non-uniform base uplift can cause floating roof jamming and damage.

(5) Anchorage points should be spaced evenly around the circumference of the tank, insofar as this is possible.

(6) The design of the holding-down bolts, straps or threaded rods should meet the requirements of EN 1993‑1‑8. The potential for corrosion of this anchorage should be considered.

(7) The design of the anchorage should accommodate movements of the tank due to thermal changes and hydrostatic pressure and minimize any stresses induced in the shell.

(8) The design of the shell for local anchorage forces and bending moments resulting from the anchorage should meet the requirements of 8.9 and 8.10 of prEN 1993‑4‑1:2024.

(9) No initial tension should be applied to the holding down bolt or strap, to ensure that it will become effective only if an uplift force develops in the shell of the tank.

NOTE If the holding down bolts or straps are not pre-tensioned, the maximum uplift forces in them under wind load will be reduced, so that the calculation described in (2) will be applicable. In addition, the stresses caused by the restraint of radial movements due to thermal changes and hydrostatic pressure will be reduced.

# Ultimate limit states in pedestal tanks

## Structural forms

(1) The provisions defined in this clause cover the design of pedestal tanks that can have the following features (Figure 13.1):

— conical storage vessel;

— tori-spherical dome with tori-conical storage vessel;

— cylindrical support tower;

— conical (tapered) tower.

|  |  |
| --- | --- |
|  |  |
| **a) Pedestal tank with conical form** | **b) Pedestal tank with tori-spherical and tori-conical form** |

Key

|  |  |
| --- | --- |
| 1 | spherical dome |
| 2 | toroidal knuckle |
| 3 | conical shell |
| 4 | toroidal transition |
| 5 | cylindrical or conical tower |

Figure 13.1 — Pedestal tank structural forms

## Actions on pedestal tanks

(1) The liquid pressures acting on the storage vessel should be taken from EN 1991‑4.

(2) The wind loads on these structures should be taken from EN 1991‑1‑4.

NOTE Additional advice on loading on these tower structures can be found in EN 1993‑3.

## Design of conical segments

(1) The design for the resistance of the conical storage vessel (Figure 13.1a) should be undertaken using the provisions of EN 1993‑1‑6 for conical shells under both internal pressure and axial compression.

NOTE The stress state in a liquid filled conical vessel is given in Annex A of prEN 1993‑1‑6:2023.

(2) The critical zone in the conical shell is at the small dimension. Considerable care should be taken with the local design of the junction between the cone and the cylindrical support.

NOTE An extensive set of experiments on the buckling of filled conical shells was undertaken in the 1960s. The outcome of the resulting design processes can be found in [9].

## Design of spherical and toroidal segments

(1) The design for the resistance of the spherical storage vessel with its toroidal transitions (Figure 13.1 b) should be undertaken using the GMNIA computational provisions of EN 1993‑1‑6.

NOTE Useful advice on the design of the tori-spherical segment is given in [6].

(2) The critical zone in the tori-conical shell is at the lower small dimension. Considerable care should be taken with the local design of this junction between the torus and the cylindrical support.

There is very little technical literature on the design of toroidal shells with negative Gaussian curvature. It is recommended that such a design should be undertaken using GMNIA according to EN 1993‑1‑6. An appropriate choice of assumed imperfections is not defined in EN 1993‑1‑6.

## Tower design

(1) The design of cylindrical and conical support towers should be undertaken using the provisions of EN 1993‑1‑6, accounting for both axial forces and global bending.

(2) Additional relevant information on the design of cylindrical tower structures can be found in EN 1993‑3.

# Serviceability limit states

## Cylindrical shell wall

(1) The serviceability limit states for cylindrical plated shell walls should be taken as:

— deformations and deflections that adversely affect the effective use of the structure;

— deformations, deflections or vibrations that cause damage to non-structural elements.

(2) Deformations, deflections and vibrations should be limited to meet the above criteria.

(3) Specific limiting values, appropriate to the intended use, should be agreed between the designer, the client and the relevant authority, taking account of the intended use and the nature of the liquids to be stored.

## Tank roofs

(1) The serviceability limit states for tank roofs should be taken as follows:

— deformations and deflections that adversely affect the effective use of the structure;

— deformations, deflections or vibrations that cause damage to non-structural elements.

(2) Deformations, deflections and vibrations should be limited to meet the above criteria.

(3) Specific limiting values, appropriate to the intended use, should be agreed between the designer, the client and the relevant authority, taking account of the intended use and the nature of the liquids to be stored.

Bibliography

**References contained in recommendations (i.e. through “should” clauses)**

The following documents are referred to in the text in such a way that some or all of their content, although not requirements strictly to be followed, constitutes highly recommended choices or course of action of this document. Subject to national regulation and/or any relevant contractual provisions, alternative standards could be used/adopted where technically justified. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 1998‑4, *Eurocode 8 — Design of structures for earthquake resistance — Part 4: Silos, tanks, pipelines, towers, masts and chimneys*

EN 10025 (all parts), Hot rolled products of structural steels

EN 10028 (all parts), Flat products made of steels for pressure purposes

EN 10029, Hot-rolled steel plates 3 mm thick or above — Tolerances on dimensions and shape

EN 10088‑4, Stainless steels — Part 4: Technical delivery conditions for sheet/plate and strip of corrosion resisting steels for construction purposes

EN 10088‑5, Stainless steels — Part 5: Technical delivery conditions for bars, rods, wire, sections and bright products of corrosion resisting steels for construction purposes

EN 10149 (all parts), Hot rolled flat products made of high yield strength steels for cold forming

EN 10216 (all parts), Seamless steel tubes for pressure purposes — Technical delivery conditions

EN 10217 (all parts), Welded steel tubes for pressure purposes — Technical delivery conditions

EN 10272, Stainless steel bars for pressure purposes

EN 14015, Specification for the design and manufacture of site built, vertical, cylindrical, flat-bottomed, above ground, welded, steel tanks for the storage of liquids at ambient temperature and above

EN 14620‑2, Design and manufacture of site built, vertical, cylindrical, flat-bottomed steel tanks for the storage of refrigerated, liquefied gases with operating temperatures between 0 °C and -165 °C — Part 2: Metallic components

EN 10028‑7, Flat products made of steels for pressure purposes — Part 7: Stainless steels

EN 10222 (all parts), Steel forgings for pressure purposes

**References contained in permissions (i.e. through “may” clauses)**

The following documents are referred to in the text in such a way that some or all of their content, although not requirements strictly to be followed, expresses a course of action permissible within the limits of the Eurocodes. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 1992 (all parts), Eurocode 2 — Design of concrete structures

EN 1997 (all parts), Eurocode 7 — Geotechnical design

EN 1999‑1‑5, Eurocode 9 — Design of aluminium structures — Part 1-5: Shell structures

EN 13445‑3, Unfired pressure vessels — Part 3: Design

**References contained in possibilities (i.e. “can” clauses) and notes**

The following documents are cited informatively in the document, for example in “can” clauses and in notes.

ISO 1000, SI units and recommendations for the use of their multiples and of certain other units

EN 12285‑1, Workshop fabricated steel tanks — Part 1: Horizontal cylindrical single skin and double skin tanks for the underground storage of flammable and nonflammable water polluting liquids other than for heating and cooling of buildings

ISO 3898, Bases for design of structures — Names and symbols of physical quantities and generic quantities

ISO 3930, General principles on reliability for structures — Vocabulary

**Other references**

The following documents are those not included in the above categories but are cited informatively in the document, for example in notes.

[1] Knödel, P., Taras, A., Ummenhofer, T.: *Low Cycle Fatigue of Shell-to-Base Joints in Storage Tanks during Operation*. TÜV Süd Tagung Flachbodentanks, Hamburg, 26.09.2019.

[2] Prinz, G.S and Nussbaumer, A, *On the low-cycle fatigue capacity of unanchored steel liquid storage tank shell-to-base connections*. Bulletin of Earthquake Engineering 10(6), 2012.

[3] Chen, L., Rotter, J.M. and Doerich, C. (2011) “*Buckling of cylindrical shells with stepwise variable wall thickness under uniform external pressure*”, Engineering Structures, Vol. 33 (12), Dec, pp. 3570-3578.

[4] Chen, L. and Rotter, J.M. (2012) “*Buckling of Anchored Cylindrical Shells of Uniform Thickness under Wind Load*”, Engineering Structures, Vol. 41, pp. 199-208.

[5] Herber, K.-H (1966) “*Vorschlag von Berechnungsgrundlagen für Beul- und Traglasten von Schalen*”, Stahlbau 35, Heft 5, pp. 142-151.

[6] Wunderlich, W. (2013) “*Toriconical and torispherical shells under uniform external or internal pressure*”, Chapter 16 in Rotter, J.M. and Schmidt, H. (eds) (2013) “*Stability of Steel Shells: European Design Recommendations: Fifth Edition Revised Impression 2013*”, Publication P125-2, European Convention for Constructional Steelwork, Brussels.

[7] Rotter, J.M. (2001) *Guide for the Economic Design of Circular Metal Silos*, Spon, London.

[8] DASt-Richtlinie 017, Entwurf: *Beulsicherheitsnachweise für Schalen — Spezielle Fälle*, Deutscher Ausschuss für Stahlbau DASt (Herausgeber), 1992.

[9] Lagae, G., Guggenberger, W. and Vanlaere, W. (2013) “*Liquid filled conical shells supported from below*”, Chapter 14 in Rotter, J.M. and Schmidt, H. (eds) (2013) “*Stability of Steel Shells: European Design Recommendations: Fifth Edition Revised Impression 2013*”, Publication P125-2, European Convention for Constructional Steelwork, Brussels.

1. All pressures are in bar gauge unless otherwise specified. [↑](#footnote-ref-1)
2. 2 As impacted by EN 1990:2023/prA1:2024 [↑](#footnote-ref-2)