The European Union

🤝 EDICT OF GOVERNMENT 🤝

In order to promote public education and public safety, equal justice for all, a better informed citizenry, the rule of law, world trade and world peace, this legal document is hereby made available on a noncommercial basis, as it is the right of all humans to know and speak the laws that govern them.

BLANK PAGE

EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

EN 1993-4-2

February 2007

English Version

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

Eurocode3-Calculdesstructuresenacier-Partie4-2:
Ré servoirs

Eurocode 3 - Bemessung und Konstruktion von
Stahlbauten - Teil 4-2: Silos,Tankbauwerke und
Rohrleitungen - Tankbauwerke

This European Standard was approved by CEN on 12 June 2006.

CEN members are bound to comply with the CEN/CENELEC Internal Regulations which stipulate the conditions for giving this European Standard the status of a national standard without any alteration. Up-to-date lists and bibliographical references concerning suchnational standards may be obtained on application to the CEN Management Centre or to any CEN member.

This European Standard exists in three official versions (English, French, German). A version in any other language made by translation under the responsibility of a CEN member into its own language and notified to the CEN Management Centre has the same status as the official versions.

CEN members are the national standards bodies of Austria, Belgium, Bulgaria, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Contents

Foreword

This European Standard EN 1993-4-2, Eurocode 3:"Design of Steel Structures – Part 4-2: Tanks", has been prepared by Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

This European Standard shall be given the status of a National Standard, either by publication of an identical text or by endorsement, at the latest by August 2007, and conflicting National Standards shall be withdrawn at latest by March 2010.

This Eurocode supersedes ENV1993-4-2: 1999.

According to the CEN-CENELEC Internal Regulations, the National Standard Organizations of the following countries are bound to implement this European Standard: Austria, Belgium, Bulgaria, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Background of the Eurocode programme

In 1975, the Commission of the European Community decided on an action programme in the field of construction, based on article 95 of the Treaty. The objective of the programme was the elimination of technical obstacles to trade and the harmonisation of technical specifications.

Within this action programme, the Commission took the initiative to establish a set of harmonised technical rules for the design of construction works which, in a first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.

For fifteen years, the Commission, with the help of a Steering Committee with Representatives of Member States, conducted the development of the Eurocodes programme, which led to the first generation of European codes in the 1980's.

In 1989, the Commission and the Member States of the EU and EFTA decided, on the basis of an agreement[1] between the Commission and CEN, to transfer the preparation and the publication of the Eurocodes to the CEN through a series of Mandates, in order to provide them with a future status of European Standard (EN). This links de facto the Eurocodes with the provisions of all the Council's Directives and/or Commission's Decisions dealing with European standards (e.g. the Council Directive 89/106/EEC on construction products - CPD - and Council Directives 93/37/EEC, 92/50/EEC and 89/440/EEC on public works and services and equivalent EFTA Directives initiated in pursuit of setting up the internal market).

The Structural Eurocode programme comprises the following standards generally consisting of a number of Parts:

EN1990   Eurocode 0: Basis of structural design
EN1991   Eurocode 1: Actions on structures
EN1992   Eurocode 2: Design of concrete structures

---

[1]   Agreement between the Commission of the European Communities and the European Committee for Standardisation (CEN) concerning the work on EUROCODES for the design of building and civil engineering works (BC/CEN/03/89).

EN1993	Eurocode 3: Design of steel structures
EN1994	Eurocode 4: Design of composite steel and concrete structures
EN1995	Eurocode 5: Design of timber structures
EN1996	Eurocode 6: Design of masonry structures
EN1997	Eurocode 7: Geotechnical design
EN1998	Eurocode 8: Design of structures for earthquake resistance
EN1999	Eurocode 9: Design of aluminium structures

Eurocode standards recognise the responsibility of regulatory authorities in each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level where these continue to vary from State to State.

Status and field of application of Eurocodes

The Member States of the EU and EFTA recognise that EUROCODES serve as reference documents for the following purposes:

- as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 - Mechanical resistance and stability - and Essential Requirement N°2 - Safety in case of fire ;
- as a basis for specifying contracts for construction works and related engineering services ;
- as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

The Eurocodes, as far as they concern the construction works themselves, have a direct relationship with the Interpretative Documents[2] referred to in Article 12 of the CPD, although they are of a different nature from harmonised product standards[3]. Therefore, technical aspects arising from the Eurocodes work need to be adequately considered by CEN Technical Committees and/or EOTA Working Groups working on product standards with a view to achieving full compatibility of these technical specifications with the Eurocodes.

The Eurocode standards provide common structural design rules for everyday use for the design of whole structures and component products of both a traditional and an innovative nature. Unusual forms of construction or design conditions are not specifically covered and additional expert consideration will be required by the designer in such cases.

National Standards implementing Eurocodes

The National Standards implementing Eurocodes will comprise the full text of the Eurocode (including any annexes), as published by CEN, which may be preceded by a National title page and National foreword, and may be followed by a National Annex.

---

[2]   According to Art. 3.3 of the CPD, the essential requirements (ERs) shall be given concrete form in interpretative documents for the creation of the necessary links between the essential requirements and the mandates for harmonised ENs and ETAGs/ETAs.

[3]   According to Art. 12 of the CPD the interpretative documents shall :

a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;

b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design. etc. ;

c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.

The Eurocodes, de facto, play a similar role in the field of the ER 1 and a part of ER 2.

The National Annex may only contain information on those parameters which are left open in the Eurocode for national choice, known as Nationally Determined Parameters, to be used for the design of buildings and civil engineering works to be constructed in the country concerned, i.e. :

- values and/or classes where alternatives are given in the Eurocode,
- values to be used where a symbol only is given in the Eurocode,
- country specific data (geographical, climatic, etc), e.g. snow map,
- the procedure to be used where alternative procedures are given in the Eurocode.

It may also contain:

- decisions on the application of informative annexes,
- references to non-contradictory complementary information to assist the user to apply the Eurocode.

Links between Eurocodes and harmonised technical specifications (ENs and ETAs) for products

There is a need for consistency between the harmonised technical specifications for construction products and the technical rules for works[4]. Furthermore, all the information accompanying the CE Marking of the construction products which refer to Eurocodes should clearly mention which Nationally Determined Parameters have been taken into account.

Additional information specific to EN1993-4-2

EN 1993-4-2 gives design guidance for the structural design of tanks.

EN 1993-4-2 gives design rules that supplement the generic rules in the many parts of EN 1993-1.

EN 1993-4-2 is intended for clients, designers, contractors and relevant authorities.

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.

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.

Safety factors for 'product type' tanks (factory production) can be specified by the appropriate authorities. When applied to 'product type' tanks, the factors in 2.9 are for guidance purposes only. They are provided to show the likely levels needed to achieve consistent reliability with other designs.

National Annex for EN1993-4-2

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-4-2 should have a National Annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1993-4-2 through:

- 2.2 (1)
- 2.2 (3)

---

[4] see Art.3.3 and Art.12 of the CPD, as well as clauses 4.2, 4.3.1, 4.3.2 and 5.2 of ID 1.

-   2.9.2.1 (1)P
-   2.9.2.1 (2)P
-   2.9.2.1 (3)P
-   2.9.2.2 (3) P
-   2.9.3 (2)
-   3.3 (3)
-   4.1.4 (3)
-   4.3.1 (6)
-   4.3.1 (8)

1   General

1.1   Scope

(1)   Part 4.2 of Eurocode 3 provides principles and application rules for the structural design of vertical cylindrical ⟨AC₁⟩ and rectangular ⟨AC₁⟩ above ground steel tanks for the storage of liquid products with the following characteristics

    a) characteristic internal pressures above the liquid level not less than −100mbar and not more than 500mbar [1];

    b) design metal temperature in the range of −50°C to +300°C. For tanks constructed using austenitic stainless steels, the design metal temperature may be in the range of −165°C to +300°C. For fatigue loaded tanks, the temperature should be limited to T < 150°C;

    c) maximum design liquid level not higher than the top of the ⟨AC₁⟩ cylindrical and rectangular tank ⟨AC₁⟩.

(2)   This Part 4.2 is concerned only with the requirements for resistance and stability of steel tanks. Other design requirements are covered by EN 14015 for ambient temperature tanks and by EN 14620 for cryogenic tanks, and by EN 1090 for fabrication and erection considerations. These other requirements include foundations and settlement, fabrication, erection and testing, functional performance, and details like man-holes, flanges, and filling devices.

(3)   Provisions concerning the special requirements of seismic design are provided in EN 1998-4 (Eurocode 8 Part 4 "Design of structures for earthquake resistance: Silos, tanks and pipelines"), which complements the provisions of Eurocode 3 specifically for this purpose.

(4)   The design of a supporting structure for a tank is dealt with in EN 1993-1-1.

(5)   The design of an aluminium roof structure on a steel tank is dealt with in EN 1999-1-5.

(6)   Foundations in reinforced concrete for steel tanks are dealt with in EN 1992 and EN 1997.

(7)   Numerical values of the specific actions on steel tanks to be taken into account in the design are given in EN 1991-4 "Actions on Silos and Tanks". Additional provisions for tank actions are given in annex A to this Part 4.2 of Eurocode 3.

(8)   This Part 4.2 does not cover:

    − floating roofs and floating covers;

    − resistance to fire (refer to EN 1993-1-2).

(9)   The circular planform tanks covered by this standard are restricted to axisymmetric structures, though they can be subject to unsymmetrical actions, and can be unsymmetrically supported.

1.2   Normative references

This European Standard incorporates, by dated and undated reference, provisions from other standards. These normative references are cited at the appropriate places in the text and the publications are listed hereafter. For dated references, subsequent amendments to, or revisions of, any of these publications apply to the European Standard only when incorporated in it by amendment or revision. For undated references the latest edition of the publication referred to applies.

---

[1] All pressures are in mbar gauge unless otherwise specified

EN 1090-2   *Execution of steel and aluminium structures – Technical requirements for steel structures*

EN 1990   *Eurocode: Basis of structural design;*

EN 1991   *Eurocode 1: Actions on structures;*

      Part 1.1: *Actions on Structures - Densities, self weight and imposed loads for buildings;*

      Part 1.2: *Actions on structures - Actions on structures exposed to fire;*

      Part 1.3: *Actions on structures - Snow loads;*

      Part 1.4: *Actions on structures - Wind loads;*

      Part 4:   *Actions on silos and tanks;*

EN 1992   *Eurocode 2 : Design of concrete structures ;*

EN 1993   *Eurocode 3: Design of steel structures;*

      Part 1.1: *General rules and rules for buildings;*

      Part 1.3: *General rules - Supplementary rules for cold formed members and sheeting;*

      Part 1.4: *General rules – Supplementary rules for stainless steels;*

      Part 1.6: *General rules - Supplementary rules for the strength and stability of shell structures;*

      Part 1.7: *General rules - Supplementary rules for planar plated structures loaded transversely;*

      Part 1.10:   *Material toughness and through thickness properties;*

      Part 4.1: *Silos;*

EN 1997   *Eurocode 7: Geotechnical design;*

EN 1998   *Eurocode 8: Design of structures for earthquake resistance;*

      Part 4:   *Silos, tanks and pipelines;*

EN 1999   *Eurocode 9: Design of aluminium structures;*

      Part 1.5: *Shell structures;*

EN 10025   [AC₁] *Hot rolled products of structural steels* [AC₁];

EN 10028   *Flat products made of steel for pressure purposes;*

EN 10088   *Stainless steels*

EN 10149   *Specification for hot-rolled flat products made of high yield strength steels for cold forming.*

      Part 1:   *General delivery conditions*

      Part 2:   *Delivery conditions for thermomechanically rolled steels*

      Part 3:   *Delivery conditions for normalized or normalized rolled steels*

EN 13084   *Freestanding industrial chimneys*

      Part 7:   *Product specification of cylindrical steel fabrications for use in single wall steel chimneys and steel liners*

EN 14015   *Specification for the design and manufacture of site built, vertical, cylindrical, flat bottomed, above ground, welded, metallic tanks for the storage of liquids at ambient temperatures*

EN 14620   *Design and manufacture of site built, vertical, cylindrical, flat-bottomed steel tanks for the storage of refrigerated, liquefied gases with operating temperatures between –5°C and –165°C;*

ISO 1000   *SI Units;*

ISO 3898   *Bases for design of structures – Notation – General symbols;*

ISO 8930   *General principles on reliability for structures - List of equivalent terms.*

1.3   Assumptions

(1)   In addition to the general assumptions of EN 1990 the following assumption applies:

- fabrication and erection complies with EN 1090, EN 14015 and 14620 as appropriate

1.4   Distinction between principles and application rules

(1)   See 1.4 in EN 1990.

1.5   Terms and definitions

(1)   The terms that are defined in 1.5 in EN 1990 for common use in the Structural Eurocodes and the definitions given in ISO 8930 apply to this Part 4.2 of EN 1993, unless otherwise stated, but for the purposes of this Part 4.2 the following supplementary definitions are given:

1.5.1   shell. A structure formed from a curved thin plate. This term also has a special meaning for tanks: see ⒶⒸ₁⟩ 1.5.9 ⟨ⒶⒸ₁.

1.5.2   axisymmetric shell. A shell structure whose geometry is defined by rotation of a meridional line about a central axis.

1.5.3   box. A structure formed from an assembly of flat plates into a three-dimensional enclosed form. For the purposes of this standard, the box has dimensions that are generally comparable in all directions.

1.5.4   meridional direction. The tangent to the tank wall at any point in a plane that passes through the axis of the tank. It varies according to the structural element being considered.

1.5.5   circumferential direction. The horizontal tangent to the tank wall at any point. It varies around the tank, lies in the horizontal plane and is tangential to the tank wall irrespective of whether the tank is circular or rectangular in plan.

1.5.6   middle surface. This term is used to refer to both the stress-free middle surface when a shell is in pure bending and the middle plane of a flat plate that forms part of a box.

1.5.7   separation of stiffeners. The centre to centre distance between the longitudinal axes of two adjacent parallel stiffeners.

Supplementary to Part 1 of EN 1993 (and Part 4 of EN 1991), for the purposes of this Part 4.2, the following terminology applies:

1.5.8 tank. A tank is a vessel for storing liquid products. In this standard it is assumed to be prismatic with a vertical axis (with the exception of the tank bottom and roof parts).

1.5.9 shell. The shell is the cylindrical wall of the tank of circular planform. Although this usage is slightly confusing when it is compared to the definition given in ⟦AC₁⟧ 1.5.1 ⟨AC₁⟩, it is so widely used with the two meanings that both have been retained here. Where any confusion can arise, the alternative term "cylindrical wall" is used.

1.5.10 tank wall. The metal plate elements forming the vertical walls, roof or a hopper bottom are referred to as the tank wall. This term is not restricted to the vertical walls.

1.5.11 course. The cylindrical wall of the tank is formed making horizontal joints between a series of short cylindrical sections, each of which is formed by making vertical joints between individual curved plates. A short cylinder without horizontal joints is termed a course.

1.5.12 hopper. A hopper is a converging section towards the bottom of a tank. It is used to channel fluids towards a gravity discharge outlet (usually when they contain suspended solids).

1.5.13 junction. A junction is the point at which any two or more shell segments or flat plate elements meet. It can include a stiffener or not: the point of attachment of a ring stiffener to the shell or box may be treated as a junction.

1.5.14 transition junction. The transition junction is the junction between the vertical wall and a hopper. The junction can be at the base of the vertical wall or part way down it.

1.5.15 shell-roof junction. The shell-roof junction is the junction between the vertical wall and the roof. It is sometimes referred to as the eaves junction, though this usage is more common for solids storages.

1.5.16 stringer stiffener. A stringer stiffener is a local stiffening member that follows the meridian of a shell, representing a generator of the shell of revolution. It is provided to increase the stability, or to assist with the introduction of local loads or to carry axial loads. It is not intended to provide a primary load carrying capacity for bending due to transverse loads.

1.5.17 rib. A rib is a local member that provides a primary load carrying path for loads causing bending down the meridian of a shell or flat plate, representing a generator of the shell of revolution or a vertical stiffener on a box. It is used to distribute transverse loads on the structure by bending action.

1.5.18 ring stiffener. A ring stiffener is a local stiffening member that passes around the circumference of the structure at a given point on the meridian. 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. In a shell of revolution it is circular, but in rectangular structures is takes the rectangular form of the plan section.

1.5.19 base ring. A base ring is a structural member that passes around the circumference of the structure at the base and is required to ensure that the assumed boundary conditions are achieved in practice.

1.5.20 ring girder or ring beam. A ring girder or ring beam is a circumferential stiffener which has bending stiffness and strength both in the plane of the circular section of a shell or the plan section of a rectangular structure and also normal to that plane. It is a primary load-carrying element, used to distribute local loads into the shell or box structure.

1.5.21 continuously supported. A continuously supported tank is one in which all positions around the circumference are supported in an identical manner. Minor departures from this condition (e.g. a small opening) need not affect the applicability of the definition.

1.5.22 discrete support. A discrete support is a [AC1> situation <AC1] in which a tank is supported using a local bracket or column, giving a limited number of narrow supports around the tank circumference.

1.5.23 catch basin. An external tank structure to contain fluid that may escape by leakage or accident from the primary tank. This type of structure is used where the primary tank contains toxic or dangerous fluids.

1.6 Symbols used in Part 4.2 of Eurocode 3

The symbols used are based on ISO 3898:1987.

1.6.1 Roman upper case letters

A	area of cross-section
A_1, A_2	area of top, bottom flange of roof centre ring
D	diameter of tank
E	Young's modulus
H	height of part of shell wall to liquid surface; maximum design liquid height
H_0	height of the tank shell
I	second moment of area of cross-section
K	coefficient for buckling design
L	height of shell segment or stiffener shear length
M	bending moment in structural member
N	axial force in structural member
N_f	minimum number of load cycles relevant for fatigue
P	vertical load on roof rafter
R	radius of curvature of shell which is not cylindrical
T	temperature
W	elastic section modulus; weight

1.6.2 Roman lower case letters

a	side length of a rectangular opening in the shell
b	side length of a rectangular opening in the shell; width of a plate element in a cross-section
c_p	coefficient for wind pressure loading
d	diameter of manhole or nozzle
e	distance of outer fibre of beam to beam axis
f_y	design yield strength of steel
f_u	ultimate strength of steel
h	rise of roof (height of apex of a dome roof above the plane of its junction to the tank shell) height of each course in tank shell
j	joint efficiency factor; stress concentration factor; count of shell wall courses
l	height of shell over which a buckle may form
m	bending moment per unit width
n	membrane stress resultant number of rafters in circular tank roof
p	distributed loading (not necessarily normal to wall)
p_n	pressure normal to tank wall (outward)
r	radius of middle surface of cylindrical wall of tank
t	wall thickness

w minimum width of base ring annular plate

x radial coordinate for a tank roof

y local vertical coordinate for a tank roof; replacement factor used in design of reinforced openings

z global axial coordinate

       coordinate along the vertical axis of an axisymmetric tank (shell of revolution)

1.6.3 Greek letters

α slope of roof

β inclination of tank bottom to vertical; $= \pi/n$ where n is the number of rafters

γ_F partial factor for actions

γ_M partial factor for resistance

δ deflection

Δ change in a variable

ν Poisson's ratio

θ circumferential coordinate around shell

σ direct stress

τ shear stress

1.6.4 Subscripts

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

F   at half span; action

a   annular

d   design value

f   fatigue

i   inside; inward directed; counting variable

k   roof centre ring

k   characteristic value

m   mean value

min   minimum allowed value

n   nominal; normal to the wall

o   outside; outward directed

p   pressure

r   radial; ring

R   resistance

s   at support

s   shell wall

x   meridional; radial; axial

y   circumferential; transverse; yield

0   reference value

1   upper

2   lower

θ   circumferential (shells of revolution)

1.7 Sign conventions

1.7.1 Conventions for global tank structure axis system for circular tanks

(1)   The sign convention given here is for the complete tank structure, and recognises 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 1.1) as follows:

Coordinate system

Coordinate along the central axis of a shell of revolution z

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 shell [AC₁⟩ (outward pressure positive) ⟨AC₁] p_n
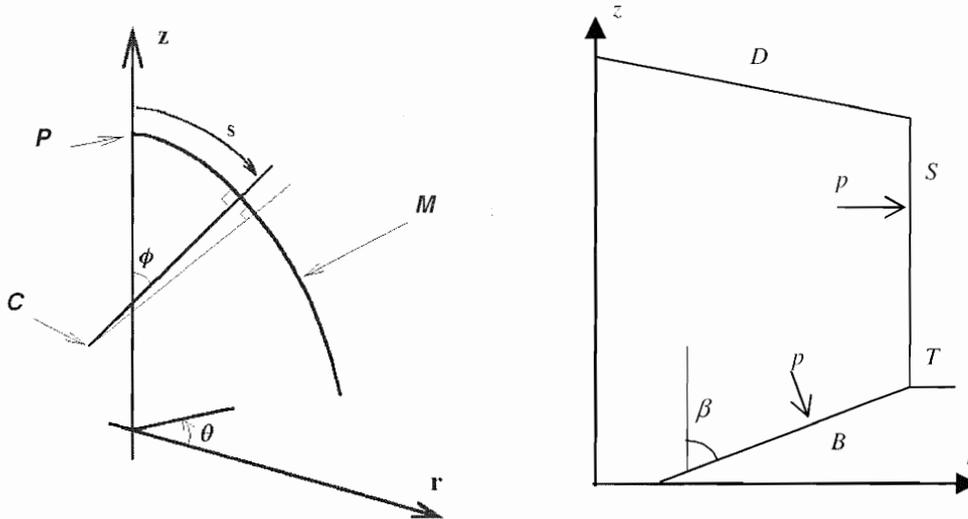
P= pole; M= shell meridian; C= Instantaneous centre of meridional curvature

D= roof; S= shell; B= bottom; T= transition

a) 3D sketch [AC₁⟩ **with global** ⟨AC₁] **axisymmetric shell coordinate system**

b) coordinates and loading: vertical section

Figure 1.1:   Coordinate systems for a circular tank

1.7.2 Conventions for global tank structure axis system for rectangular tanks

(1)   The sign convention given here is for the complete tank structure, and recognises 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 box 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 Cartesian coordinates x, y, z, where the vertical direction is taken as z (see figure 1.2).

(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)

Shear stresses: see [AC₁⟩ 1.7.4 ⟨AC₁]

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

Pressure normal to box (outward positive) p
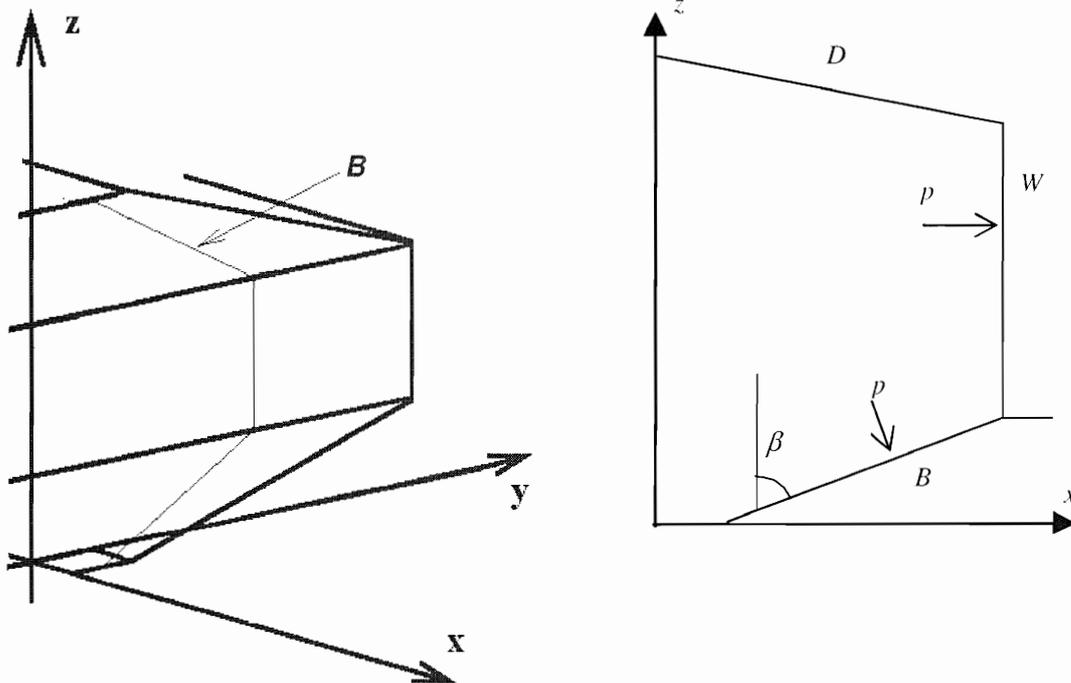
| B= box meridian | D= roof; W= wall; B= bottom |
| a) **3D sketch** [AC₁> **with global** [AC₁] **coordinate system** | b) **coordinates and loading: vertical section** |

Figure 1.2:   Coordinate systems for a rectangular tank

1.7.3 Conventions for structural element axes in both circular and rectangular tanks

(1)   The convention for structural elements attached to the tank wall (see figures 1.3 and 1.4) is different for meridional and circumferential members.

(2)   The convention for meridional straight structural elements (see figure 1.3a) attached to the tank wall (for both a shell and a box) is:

Meridional coordinate for cylinder, hopper and roof attachment x

Strong bending axis (parallel to flanges) y

Weak bending axis (perpendicular to flanges) z

a) stiffener and axes of bending

b) local axes in different segments

D= roof, S= shell, B=bottom

Figure 1.3:   Local coordinate systems for meridional stiffeners on a shell or box
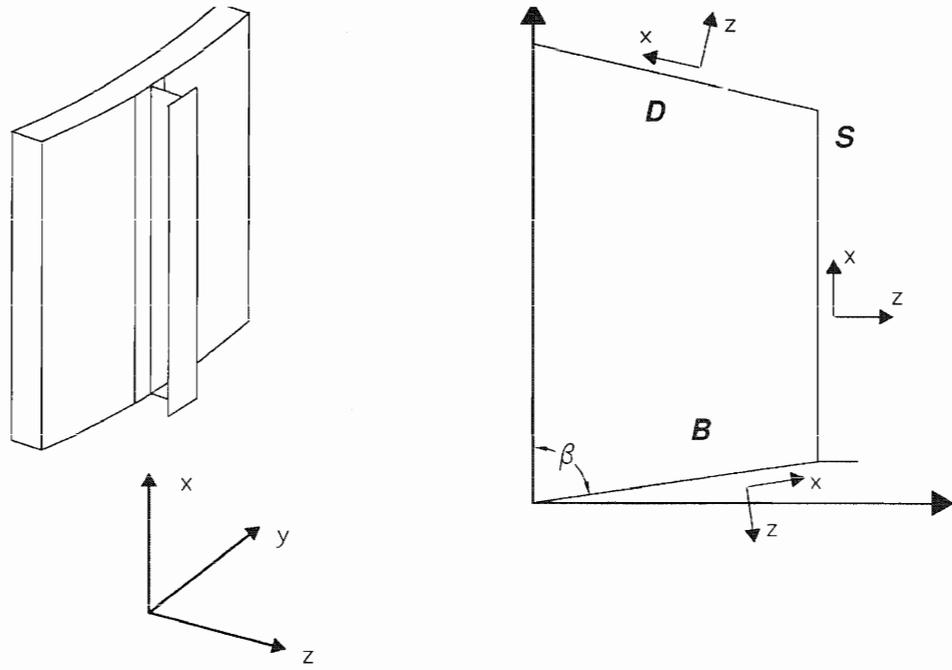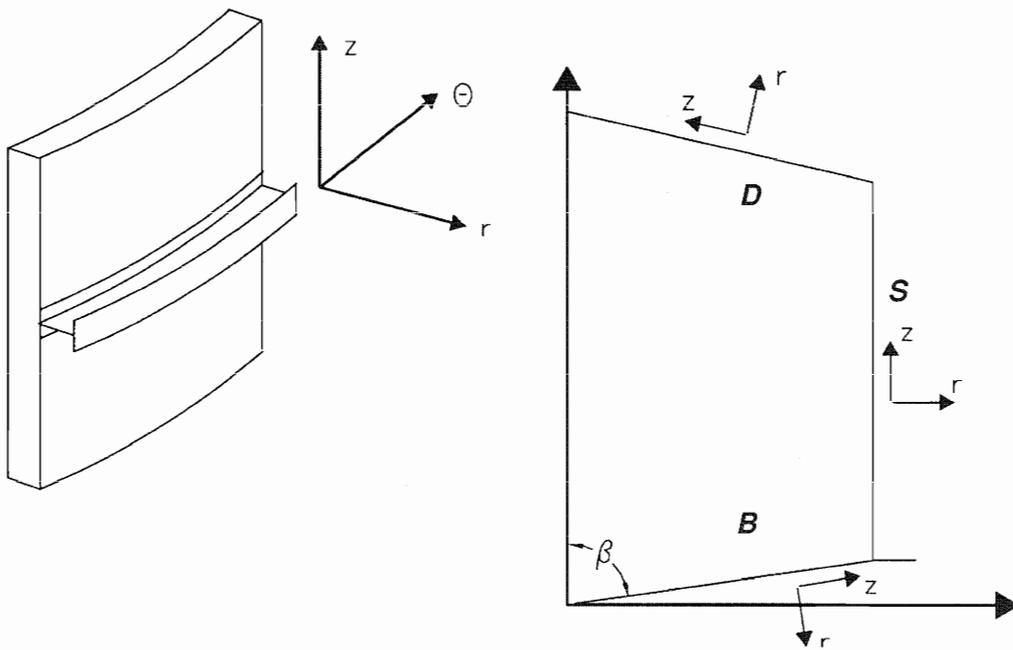
D= roof; S= shell; B= bottom

a) stiffener and axes of bending

b) local axes in different segments

Figure 1.4: Local coordinate systems for circumferential stiffeners on a shell or box

(3)   The convention for circumferential curved structural elements (see figure 1.4a) attached to a shell wall is:

Circumferential coordinate axis (curved) θ

Radial axis r

Vertical axis z

(4)   The convention for circumferential straight structural elements attached to a box is:

Circumferential axis x

Horizontal axis y

Vertical axis z

1.7.4 Conventions for stress resultants for circular tanks and rectangular 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 1.5.

Membrane stress resultants, see figure 1.5:

n_x meridional membrane stress resultant

n_θ circumferential membrane stress resultant in shells

n_y circumferential membrane stress resultant in rectangular boxes

n_{xy} or $n_{x\theta}$ membrane shear stress resultant

Membrane stresses:

σ_{mx} meridional membrane stress

$\sigma_{m\theta}$ circumferential membrane stress in shells

σ_{my} circumferential membrane stress in rectangular boxes

τ_{mxy} or $\tau_{mx\theta}$ membrane shear stress

(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 1.5.

NOTE: This plate and shell convention is at variance with beam and column conventions used in Eurocode 3: Parts 1.1 and 1.3. Care needs to be exercised when using them in conjunction with these provisions.

Bending stress resultants, see figure 1.5:

m_x meridional bending moment per unit width

m_θ circumferential bending moment per unit width in shells

[AC₁⟩ m_y circumferential bending moment per unit width in rectangular boxes ⟨AC₁]

m_{xy} or $m_{x\theta}$ twisting shear moment per unit width

Bending stresses:

σ_{bx} meridional bending stress

$\sigma_{b\theta}$ circumferential bending stress in shells

σ_{by} circumferential bending stress in rectangular boxes

τ_{bxy} or $\tau_{bx\theta}$ twisting shear stress

Inner and outer surface stresses:

σ_{six}, σ_{sox} meridional inner, outer surface stress

$\sigma_{si\theta}$, $\sigma_{so\theta}$ circumferential inner, outer surface stress in shells

σ_{siy}, σ_{soy} circumferential inner, outer surface stress in rectangular boxes

τ_{sixy}, τ_{soxy} inner, outer surface shear stress in rectangular boxes

a) **Membrane stress resultants**   b) **Bending stress resultants**
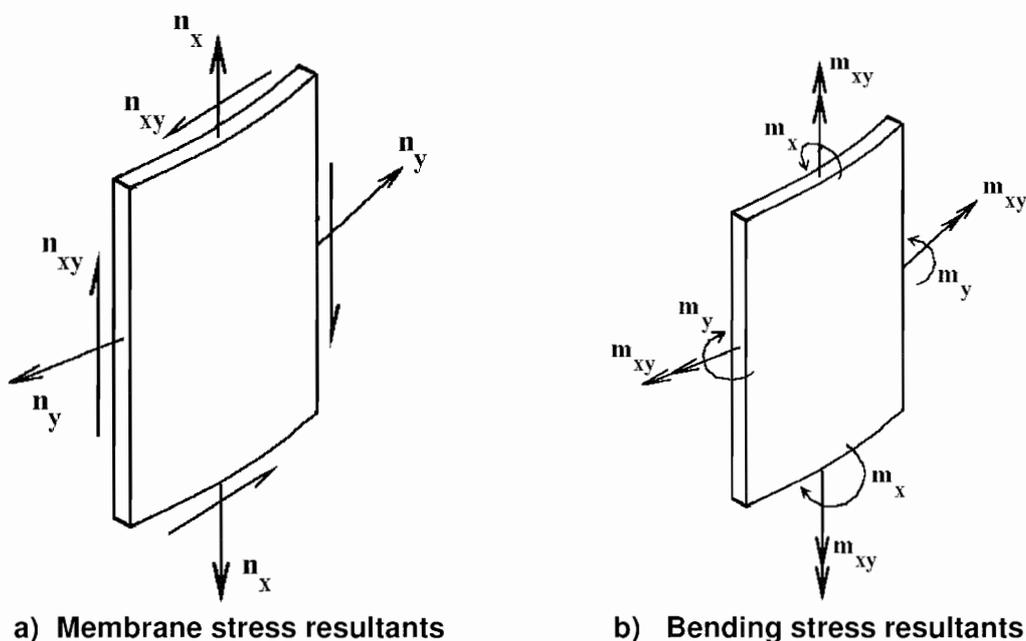
Figure 1.5: Stress resultants in the tank wall (shells and boxes)

1.8 Units

(1)P S.I. units shall be used in accordance with ISO 1000.

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

– dimensions	:	m	mm
– unit weight	:	kN/m^3	N/mm^3
– forces and loads	:	kN	N
– line forces and line loads	:	kN/m	N/mm
– pressures and area distributed actions	:	kPa	MPa
– unit mass	:	kg/m^3	kg/mm^3
– acceleration	:	km/s^2	m/s^2
– membrane stress resultants	:	kN/m	N/mm
– bending stress resultants	:	kNm/m	Nmm/mm
– stresses and elastic moduli	:	kPa	MPa ($=N/mm^2$)

18

2 Basis of design

2.1 Requirements

(1)P A tank shall be designed, constructed and maintained to meet the requirements of section 2 of EN 1990 as supplemented by the following.

(2) Special consideration should be given to situations during erection.

2.2 Reliability differentiation

(1) For reliability differentiation see EN 1990.

NOTE: The National Annex may define consequence classes for tasks as a function of the location, type of infill and loading, the structural type, size and type of operation.

⟦AC₁⟩ (2) Different levels of rigour should ⟨AC₁⟧ be used in the design of tanks, depending on the consequence class chosen, that also includes the structural arrangement and the susceptibility to different failure modes.

(3) In this Part, three consequence classes are used with requirements which produce designs with essentially equal risk in the design assessment and considering the expense and procedures necessary to reduce the risk of failure for different structures: consequence classes 1, 2 and 3.

NOTE: The National Annex may provide information on the consequence classes. The following classification is recommended.

- **Consequence Class 3**: Tanks storing liquids or liquefied gases with toxic or explosive potential and large size tanks with flammable or water-polluting liquids in urban areas. Emergency loadings should be taken into account for these structures where necessary, see annex A.2.14.

- **Consequence Class 2**: Medium size tanks with flammable or water-polluting liquids in urban areas.

- **Consequence Class 1**: Agricultural tanks or tanks containing water

(4)P The choice of the relevant Consequence Class shall be agreed between the designer, the client and the relevant authority.

2.3 Limit states

(1) The limit states defined in EN 1993-1-6 should be adopted for this Part.

2.4 Actions and environmental effects

(1)P The general requirements set out in section 4 of EN 1990 shall be satisfied.

(2) Because the information wind loads on liquid induced loads, internal pressure loads, thermally induced loads, loads resulting from pipes valves and other items connected to the tank , loads resulting from uneven settlement and emergency loadings set down in EN1991 is not complete special information is given in annex A

2.5 Material properties

(1) The general requirements for material properties given in EN 1993-1-1 should be followed.

(2)   The specific properties of materials for tanks given in section 3 of this Part should be used.

2.6   Geometrical data

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

(2)   The additional information specific to shell structures provided in EN 1993-1-6 may be used.

(3)   The plate thicknesses given in 4.1.2 should be used in calculations.

2.7   Modelling of the tank for determining action effects

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

(2)   The specific requirements for structural analysis in relation to serviceability set out in 5.5, 7.5 and 9.4 should be used for the relevant structural segments.

(3)   The specific requirements for structural analysis in relation to ultimate limit states set out in 5.3, 7.3 and 9.3 (and in more detail in EN 1993-1-6) should be applied.

2.8   Design assisted by testing

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

2.9   Action effects for limit state verifications

2.9.1 General

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

2.9.2 Partial factors for ultimate limit states

2.9.2.1   Partial factors for actions on tanks

(1)P   For persistent and transient design situations, the partial factors γ_F shall be used.

> **NOTE:** The National Annex may provide values for the (AC1) partial factors (AC1) γ_F Table 2.1 gives the recommended values for γ_F.

(2)P   For accidental design situations, the partial factors γ_F for the variable actions shall be used. This also applies to the liquid loading of catch basins.

> **NOTE:** The National Annex may provide values for the (AC1) partial factors (AC1) γ_F Table 2.1 gives the recommended values for γ_F.

 (3)P   Partial factors for 'product type' tanks (factory production) shall be specified.

> **NOTE:** The National Annex may provide values for the partial factors γ_F Table 2.1 gives the recommended values for γ_F.

Table 2.1: Recommended values for the partial factors for actions on tanks for persistent and transient design situations and for accidental design situation

design situation	liquid type	recommended values for γ_F in case of variable actions from liquids	recommended values for γ_F in case of permanent actions
liquid induced loads during operation	toxic, explosive or dangerous liquids	1,40	1,35
	flammable liquids	1,30	1,35
	other liquids	1,20	1,35
liquid induced loads during test	all liquids	1,00	1,35
accidental actions	all liquids	1,00	

2.9.2.2   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 section 9 of EN 1993-1-6.

(3)P   The partial factors γ_{Mi} shall be specified according to Table 2.2.

Table 2.2: Partial factors for resistance

Resistance to failure mode	Relevant γ
Resistance of welded or bolted shell wall to plastic limit state, cross-sectional resistance	γ_{M0}
Resistance of shell wall to stability	γ_{M1}
Resistance of welded or bolted shell wall to rupture	γ_{M2}
Resistance of shell wall to cyclic plasticity	γ_{M4}
Resistance of welded or bolted connections or joints	γ_{M5}
Resistance of shell wall to fatigue	γ_{M6}

NOTE: Partial factors γ_{Mi} for tanks may be defined in the National Annex. For values of γ_{M5}, further information may be found in EN 1993-1-8. For values of γ_{M6}, further information may be found in EN 1993-1-9. The following numerical values are recommended for tanks:

$\gamma_{M0} = 1,00$	$\gamma_{M1} = 1,10$	$\gamma_{M2} = 1,25$
$\gamma_{M4} = 1,00$	$\gamma_{M5} = 1,25$	$\gamma_{M6} = 1,10$

2.9.3 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 need not be carried out.

(2)   For all serviceability limit states the values of γ_{Mser} should be specified.

> NOTE: The National Annex may provide information on the value for the partial factor for serviceability γ_{Mser}. $\gamma_{Mser} = 1$ is recommended.

2.10 Combinations of actions

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

(2)   Imposed loads and snow loads need not be considered to act simultaneously.

(3)   Reduced wind actions, based on a short exposure period, may be used when wind is in combination with the actions of the hydrostatic test.

(4)   Seismic actions need not be considered to act during test conditions.

(5)   Emergency actions need not be considered to act during test conditions. The combination rules for accidental actions given in EN 1990 should be applied to emergency situations.

2.11 Durability

(1)   The general requirements set out in EN 1990 should be followed.

3 Properties of materials

3.1 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 this section should be treated as nominal values to be adopted as characteristic values in design calculations.

(4) Other material properties are given in the relevant Reference Standards defined in EN 1993-1-1.

(5) Where the tank may be filled with hot liquids, the values of the material properties should be appropriately reduced to values corresponding to the maximum temperatures to be encountered.

(6) The material characteristics at elevated temperature ($T > 100°C$ for structural steels and $T > 50°C$ for stainless steels) should be obtained from EN 13084-7.

3.2 Structural steels

(1) The methods for design by calculation given in this Part 4.2 of EN 1993 may be used for structural steels as defined in EN 1993-1-1, which conform with parts 2 to 6 of EN 10025. The methods may also be used for steels included in EN 1993-1-3.

(2) The mechanical properties of structural steels according to EN 10025 or $\boxed{\text{AC}_1}$ EN 10149 $\boxed{\text{AC}_1}$ should be taken from EN 1993-1-1 or EN 1993-1-3.

3.3 Steels for pressure purposes

(1) The methods for design by calculation given in this Part 4.2 of EN 1993 may be used for steels for pressure purposes conforming with EN 10028 provided that:

  − the yield strength is in the range covered by EN 1993-1-1;

  − the ultimate strain is not less than the minimum value for steels according to EN 1993-1-1 which have the same specified yield strength;

  − the ratio f_u/f_y is not less than 1,10.

(2) The mechanical properties of steels for pressure purposes should be taken according to EN 10028.

(3) Where the design involves a stability calculation, appropriate reduced properties should be used, see EN 1993-1-6 section 3.1.

   NOTE: Further information may be given in the National Annex.

3.4 Stainless steels

(1) The mechanical properties of stainless steels according to EN 10088 should be obtained from EN 1993-1-4.

(2)   Guidance for the selection of stainless steels in view of corrosion actions may be obtained from appropriate sources.

(3)   Where the design involves a buckling calculation, appropriate reduced properties should be used, see EN 1993-1-6.

3.5 Toughness requirements

3.5.1 General

(1)   The toughness requirements should be determined for the minimum design metal temperature according to EN 1993-1-10.

(2)   The minimum design metal temperature MDMT should be determined according to 3.5.2. MDMT may be used in place of T_{ed}. in EN 1993-1-10.

3.5.2 Minimum design metal temperature

(1)   The minimum design metal temperature MDMT should be the lowest of the minimum temperature of the contents or those classified in table 3.1.

(2)   The lowest one day mean ambient temperature LODMAT should be taken as the lowest recorded temperature averaged over any 24 hour 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.

Table 3.1: Minimum design metal temperature MDMT based on LODMAT

Lowest one day mean ambient temperature LODMAT	Minimum design metal temperature MDMT	
	10 years data	30 years data
$-10°C \leq$ LODMAT	LODMAT $+5°C$	LODMAT $+10°C$
$-25°C \leq$ LODMAT $\leq -10°C$	LODMAT	LODMAT $+5°C$
LODMAT $\leq -25°C$	LODMAT $-5°C$	LODMAT

4   Basis for structural analysis

4.1   Ultimate limit states

4.1.1   Basis

(1)   Steel structures and components should be so proportioned that the basic design requirements given in section 2 are satisfied.

4.1.2   Plate thickness to be used in resistance calculations

(1)   In calculations to determine the resistance, the design value of thickness for a plate is the nominal thickness specified in EN 10025, EN 10028 [AC₁⟩ EN 10149 ⟨AC₁] or EN 10088 reduced by the maximum value of minus tolerance and a value of corrosion allowance specified in 4.1.3.

4.1.3   Effects of corrosion

(1)   The effects of corrosion should be taken into account.

(2)   The corrosion depends upon the stored liquid, the type of steel, the heat treatment and the measures taken to protect the construction against corrosion.

(3)   The value of an allowance should be specified if necessary.

4.1.4   Fatigue

(1)P   With frequent load cycles the structure shall be checked against the fatigue limit state.

(2)   The design against low cycle fatigue may be carried out according to EN 1993-1-6.

(3)   If variable actions will be applied with more than N_f cycles during the design life of the structure the design should be checked against fatigue (LS4) according to section 9 of EN 1993-1-6.

> **NOTE:** The National Annex may provide the value for the number N_f of cycles. The value $N_f = 10000$ is recommended.

4.1.5   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.

4.2   Analysis of the circular shell structure of a tank

4.2.1   Modelling of the structural shell

(1)   The modelling of the structural shell should follow the requirements of EN 1993-1-6, but these may be deemed to be satisfied by the following provisions.

(2)   The modelling of the structural shell should include all stiffeners, openings and attachments.

(3)   The design should ensure that the assumed boundary conditions are satisfied.

4.2.2 Methods of analysis

4.2.2.1   General

(1)   The analysis of the tank shell 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 Consequence Class.

(3)   Irrespective of the Consequence Class chosen, the simplified design described in Section 11 may be used if the conditions listed there are met.

4.2.2.2   Consequence Class 1

(1)   For tanks in Consequence Class 1, membrane theory may be used to determine the primary stresses, with factors and simplified expressions to describe local bending effects and unsymmetrical actions.

4.2.2.3   Consequence Class 2

(1)   For tanks in Consequence Class 2 under axisymmetric actions and support, one of two alternative analyses should be used:

    a)   Membrane theory may be used to determine the primary stresses, with bending theory elastic expressions to describe all local effects.

    b)   A validated numerical analysis may be used (for instance, finite element shell analysis) as defined in EN 1993-1-6.

(2)   Where the loading condition is not axisymmetric, a validated numerical 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 harmonic 1), membrane theory may be used to determine the primary stresses.

(4)   For analyses of actions due to wind loading and/or foundation settlement, semi-membrane theory or membrane theory may be used.

> NOTE: For information concerning membrane theory, see EN 1993-1-6. The semi-membrane theory describes the membrane behaviour in interaction with the circumferential bending stiffness.

(5)   Where membrane theory is used to analyse the shell, discrete rings attached to an isotropic cylindrical tank shell under internal pressure may be deemed to have an effective area which includes a length of shell above and below the ring of $0,78\sqrt{rt}$, except where the ring is at a junction.

(6)   Where the shell is discretely stiffened by vertical stiffeners, the stresses in the stiffeners and the shell wall may be calculated by treating the stiffeners as smeared on the shell wall, provided the spacing of the stiffeners is no wider than $5\sqrt{rt}$.

(7)   Where vertical stiffeners are smeared, the stress in the stiffener should be determined making proper allowance for compatibility between the stiffener and the wall and the wall stress in the orthogonal direction, according to 4.4.

(8)   If 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.

(9)   Where a ring girder is treated as a prismatic section (free of distortion), the vertical web segment should have a plate slenderness not greater than $b/t = 20$.

(10)   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.

4.2.2.4   Consequence Class 3

(1)   For tanks in Consequence Class 3, the internal forces and moments should be determined using a validated analysis (for instance, finite element shell analysis) as defined in EN 1993-1-6. The plastic limit state (LS1) may be assessed using plastic collapse strengths under primary stress states as defined in EN 1993-1-6.

4.2.3 Geometric imperfections

(1)   Geometric imperfections in the shell should satisfy the limitations defined in EN 1993-1-6.

(2)   For tanks in Consequence Classes 2 and 3, the geometric imperfections should be measured following construction to ensure that the assumed fabrication tolerance has been achieved.

(3)   Geometric imperfections in the shell need not be explicitly included in determining the internal forces and moments, except where a GNIA or GMNIA analysis is used, as defined in EN 1993-1-6.

4.3   Analysis of the box structure of a rectangular tank

4.3.1 Modelling of the structural box

(1)   The modelling of the structural box should follow the requirements of EN 1993-1-7, but they may be deemed to be satisfied by the following provisions.

(2)   The modelling of the structural box should include all stiffeners, openings and attachments.

(3)   The design should ensure that the assumed boundary conditions are satisfied.

(4)   The joints between segments of the box should satisfy the modelling assumptions for strength and stiffness.

(5)   Each panel of the box may be treated as an individual plate segment provided that both:

    a)   the forces and moments introduced into each panel by its neighbours are included;

    b)   the flexural stiffness of adjacent panels is included.

(6)   Where the wall panel is discretely stiffened by stiffeners, the stress in the stiffeners and in the wall may be calculated by treating the stiffeners as smeared on the box wall, provided that the spacing of the stiffeners is no wider than $n_S\, t$.

    NOTE: The National Annex may choose the value of n_S. The value $n_S = 40$ is recommended.

(7)   Where smeared stiffeners are used, the stress in the stiffener should be determined making proper allowance for eccentricity of the stiffener from the wall plate, and for the wall stress in the direction orthogonal to the axis of the stiffener.

(8)   The effective width of plate on each side of a stiffener should be taken as not greater than $n_{ew} t$, where t is the local plate thickness.

   NOTE: The National Annex may choose the value of n_{ew}. The value ⟨AC1⟩ $n_{ew} = 15\varepsilon$ ⟨AC1⟩ is recommended.

4.3.2 Geometric imperfections

(1)   Geometric imperfections in the box should satisfy the limitations defined in EN 1993-1-7.

(2)   Geometric imperfections in the box need not be explicitly included in determining the internal forces and moments.

4.3.3 Methods of analysis

(1)   The internal forces in the plate segments of the box wall may be determined using either:

   a)   static equilibrium for membrane forces and beam theory for bending;

   b)   an analysis based on linear plate bending and stretching theory;

   c)   an analysis based on nonlinear plate bending and stretching theory.

(2)   For tanks in Consequence Class 1, method (a) in (1) may be used.

(3)   Where the design loading condition is symmetric relative to each plate segment and the tank is in Consequence Class 2, method (a) in (1) may be used.

(4)   Where the loading condition is not symmetric and the tank is in Consequence Class 2, either method (b) or method (c) in (1) should be used.

(5)   For tanks in Consequence Class 3, the internal forces and moments should be determined using either method (b) or method (c) in (1).

4.4   Equivalent orthotropic properties of 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.

(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 4.4 of EN 1993-4-1.

5 Design of cylindrical walls

5.1 Basis

5.1.1 General

(1) Cylindrical shell walls should be so proportioned that the basic design requirements for the ultimate limit state given in section 2 are satisfied.

(2) The safety assessment of the cylindrical shell should be carried out using the provisions of EN 1993-1-6.

5.1.2 Wall design

(1) The cylindrical shell wall of the tank should be checked for the following phenomena under the limit states defined in EN 1993-1-6:

- Global stability and static equilibrium

- LS1: plastic limit

- LS2: cyclic plasticity

- LS3: buckling

- LS4: fatigue

(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) For tanks in Consequence Class 1, the cyclic plasticity and fatigue limit states may be ignored.

5.2 Distinction of cylindrical shell forms

(1) A cylindrical shell wall constructed from flat rolled steel sheet is termed 'isotropic' (see 5.3.2 of EN 1993-4-1).

(2) A cylindrical shell wall constructed from corrugated steel sheets where the troughs pass around the circumference of the tank is termed 'horizontally corrugated' (see 5.3.4 of EN 1993-4-1) .

(3) A cylindrical shell wall with stiffeners attached to the outside is termed 'externally stiffened' irrespective of the spacing of the stiffeners (see 5.3.3 of EN 1993-4-1).

5.3 Resistance of the tank shell wall

(1) The resistance of the cylindrical shell should be evaluated using the provisions of EN 1993-1-6, except where the clauses of 5.4 contain provisions that are deemed to satisfy the provisions of that standard.

(2) The joint efficiency of full penetration butt welds may be taken as unity provided that the requirements of EN14015 or EN14620, as appropriate, are met.

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

5.4 Considerations for supports and openings

5.4.1 Shell supported by a skirt

(1) Where the cylindrical shell is supported by a skirt, this should satisfy the provisions of EN 1993-4-1.

5.4.2 Cylindrical shell with engaged columns

(1) Where the cylindrical shell is supported with engaged columns, this should satisfy the provisions of EN 1993-4-1.

5.4.3 Discretely supported cylindrical shell

(1) Where the cylindrical shell is discretely supported by columns or other devices, the provisions of EN 1993-4-1 for this condition should be satisfied.

5.4.4 Discretely supported tank with columns beneath the hopper

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

5.4.5 Local support details and ribs for load introduction in cylindrical walls

5.4.5.1 Local supports beneath the wall of a cylinder

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

5.4.5.2 Local ribs for load introduction into cylindrical walls

(1) Local ribs for load introduction into cylindrical walls should satisfy the provisions of EN 1993-4-1.

5.4.6 Openings in tank walls

5.4.6.1 General

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

(2) This reinforcement may be achieved by:

- increasing the thickness of the shell plate;

- adding a reinforcing plate;

- the presence of 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).

5.4.6.2   Shell nozzles of small size

(1)   Shell nozzles with outside diameter less than 80mm are classed as of small size.

(2)   Reinforcement may be omitted, provided that the thickness of the wall at the nozzle is not less than that given in table 5.1.

Table 5.1: 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
$d_n \le 50$	5,0	3,5
$50 < d_n \le 75$	5,5	5,0
$75 < d_n \le 80$	7,5	6,0

5.4.6.3   Design of shell man holes and shell nozzles of large size for LS1

(1)   Shell man holes and shell nozzles with outside diameter greater than 80mm are classed as of large size.

(2)   The design may be undertaken using either the area replacement method according to paragraphs (3) and (4), or alternatively by the method described in paragraph (5) and (6).

(3)   A reinforcement of cross-sectional area ΔA should be provided in the vertical plane containing the centre of the opening, given by:

$$\Delta A = 0.75 \, d \, t_{ref} \qquad \qquad ... (5.1)$$

where:

   d is the diameter of the hole cut in the shell plate;

   t_{ref} is the thickness required by the design for LS1 for the shell plate without opening.

(4)   The reinforcing area ΔA 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 $1.5d < d_n < 2d$, where 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).

(5)   As an alternative to the area replacement method specified in (3) and (4) the reinforcement may be achieved by introducing a nozzle body that protrudes on both sides of the shell plate by an amount

not less than $1,17 \sqrt{r_m t_{ref,n}}$. This method should not be used unless the nozzle body is more than 100mm from the base ring plate.

(6)   The thickness of the nozzle body should be chosen such that the stress concentration factor j does not exceed 2,0. The stress concentration factor j should be obtained from figure 5.1 using the replacement factor y. The replacement factor y should be evaluated from:

$$y = 1,56\frac{t_n}{t}\sqrt{\frac{t_n}{r_m}} + \left(\frac{t_n}{2r_m}\right) \qquad \text{... (5.2)}$$

where:

> t is the shell plate thickness;
>
> t_n is the nozzle body thickness;
>
> r_m is the mean radius of the nozzle (nozzle middle surface);
>
> r_e is the external radius of the nozzle;
>
> r_i is the inside radius of the nozzle.
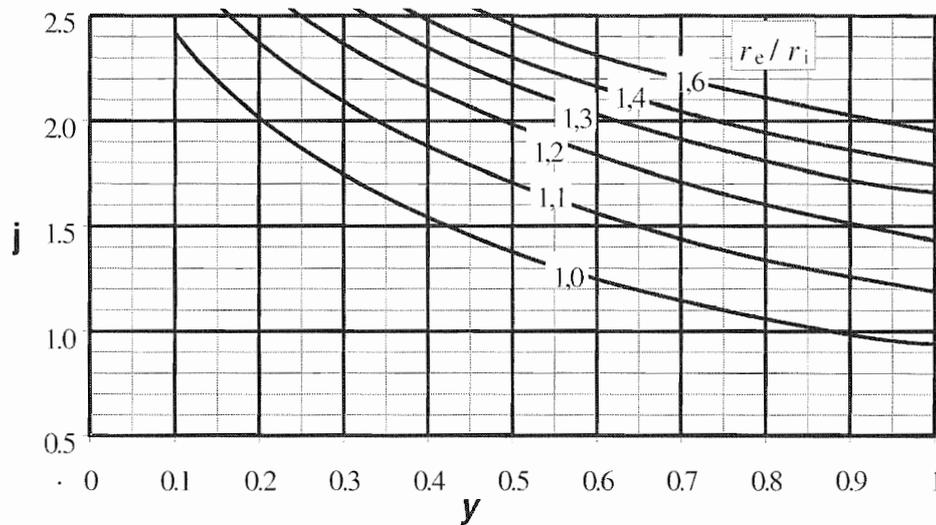
j = Stress concentration factor; y = Replacement factor

Figure 5.1: Stress concentration factor for barrel-type nozzle reinforcements

5.4.6.4   Design for LS3 in the presence of shell openings

(1)   The effect of openings on the stability of shells may be neglected provided that the dimensionless opening size η is smaller than $\eta_{max} = 0,6$, and η is given by:

$$\eta = \frac{r_0}{\sqrt{rt}} \qquad \text{... (5.3)}$$

where:

> r is the radius of the cylindrical shell near the opening;

t is the thickness of the unstiffened shell wall near the opening;

r_0 is the radius of the opening.

(2)   Where the opening is rectangular, the equivalent opening radius may be taken as:

$$r_0 = \frac{a+b}{4}$$ ... (5.4)

where:

a is the horizontal side length of the opening;

b is the vertical height of the opening.

(3)   Where the radius of the 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 cross-sectional area taken away by the opening is smaller than the reinforcement cross-sectional area ΔA. The reinforcement can be provided according to 5.4.6.3 (4) or by means of stiffeners in the meridional direction.

(4)   If stiffeners in the meridional direction are used to reinforce the opening, the cross-section of each stiffener should be reduced towards the ends to prevent the formation of buckles due to stress concentration in the shell plate near the stiffener ends.

5.4.7 Anchorage of the tank

(1)   The anchorage should be principally attached to the cylindrical shell and not to the base ring plate alone.

(2)   The design should accommodate movements of the tank due to thermal changes and hydrostatic pressure to minimise stresses induced in the shell by these effects.

(3)   Where the tank is supported on a rigid anchorage, and is subject to horizontal loads (e.g. wind, impact) the anchorage forces should be calculated according to shell theory.

> **NOTE:** It should be noted that these forces may be locally much higher than those found using beam theory. See clause (3) of section 5.4.7 of EN 1993-4-1.

(4)   The design of the cylindrical shell for local anchorage forces and bending moments resulting from the anchorage should meet the provisions of [AC₁⟩ 1993-4-1 ⟨AC₁] .

5.5 Serviceability limit states

5.5.1 Basis

(1)   The serviceability limit states for cylindrical plated 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.

6 Design of conical hoppers

(1)   The design of conical hoppers should satisfy the requirements of EN 1993-4-1.

7 Design of circular roof structures

7.1 Basis

7.1.1 General

(1)   Steel tank roofs should be so proportioned that the basic design requirements for the ultimate limit state given in section 2 are satisfied.

(2)   The safety assessment of the spherical or conical shell should be carried out using the provisions of EN 1993-1-6.

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

7.1.2 Roof design

(1)   The roof should be checked for:

  –   resistance to buckling;

  –   resistance of the joints (connections);

  –   resistance to rupture under internal pressure.

(2)   The roof plating should satisfy the provisions of EN 1993-1-6 except where 7.3 to 7.5 provide an alternative approach.

7.2 Distinction of roof structural forms

(1)   The roof may either have a spherical, a conical, a torispherical or a toriconical shape. Where high internal pressures occur above the liquid surface, the shape should preferably be chosen as torispherical or toriconical.

(2)   A roof structure in one of the shapes described in (1) may either be unsupported or supported by structural members.

(3)   The roof supporting structure according to (2) may be supported by columns.

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

(5)   The roof plating may be:

  a)   supported by the roof structure without connection;

  b)   attached to the roof structure.

(6)   Where frangibility of the roof is required, type (a) should be used.

(7)   Where the roof supporting structure is external, type (b) should be used.

7.3   Resistance of circular roofs

(1)   The roof plating should satisfy the provisions of EN 1993-1-6 unless special provisions are given in 7.4.

(2)   The roof supporting structure should satisfy the provisions of EN 1993-1-1.

(3)   Torispherical and toriconical roofs should be designed to prevent buckling of the knuckle region under internal pressure.

7.4   Considerations for individual structural forms

7.4.1   Unsupported roof structure

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

(2)   In double welded lap construction, the reduction of resistance against buckling and the plastic limit state due to the joint eccentricities should be taken into account in the model for the analysis.

7.4.2   Cone or dome roof with supporting structure

7.4.2.1   Plate design

(1)   The roof plating may be designed using large deflection theory.

(2)   Where roof frangibility is required, roof plates should not be attached to the internal roof supporting structure.

7.4.2.2   Design of the supporting structure

(1)   The roof supporting structure should satisfy the provisions of EN 1993-1-1.

(2)   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 effective width may be taken as $16t$ unless a larger value is confirmed by an analysis.

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

7.4.3   Roof to shell junction (eaves junction)

(1)   The roof to cylinder junction (eaves junction) should be designed to carry the total downward vertical load from the roof (dead weight, snow, live load and internal negative pressure).

(2)   The roof to cylinder junction should satisfy the provisions of EN 1993-1-6. If the conditions set out in 11.1 (1) are satisfied, the simplified design method given in 11.2.5 may be applied.

(3)   For frangible roof design the compression area A should satisfy the condition:

$$\boxed{\text{AC}_1}\ A \geq \frac{W}{2\pi \tan \alpha f_{yd}}\ \boxed{\text{AC}_1} \qquad\qquad \text{... (7.1)}$$

where:

  W   is the total weight of the shell and any framing (but not roof-plates) supported by the shell and roof;

  α is the angle between the roof and a horizontal plane at the roof to cylinder junction.

7.5   Serviceability limit states

(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.

8   Design of transition junctions at the bottom of the shell and supporting ring girders

(1)   The design of transition junctions at the bottom edge and supporting ring girders should satisfy the requirements of EN 1993-4-1.

9 Design of rectangular and planar-sided tanks

9.1 Basis

(1) A rectangular tank should be designed either as stiffened box in which the structural action is predominantly bending, or as a thin membrane structure in which the action is predominantly membrane stresses developing after large deformations.

(2) Where the box is designed for bending action, the joints should be designed to ensure that the connectivity assumed in the stress analysis is achieved in the execution.

9.2 Distinction of structural forms

9.2.1 Unstiffened tanks

(1) A structure that is fabricated from flat steel plates without attached stiffeners should be treated as an 'unstiffened box`.

(2) A structure that is stiffened only along joints between plates which are not coplanar should also be treated as an 'unstiffened box'.

9.2.2 Stiffened tanks

(1) A structure that is fabricated from flat plates to which stiffeners are attached within the plate area should be treated as a 'stiffened box'. The stiffeners may be circumferential or vertical or orthogonal.

9.2.3 Tanks with ties

(1) Tanks with ties may be square or rectangular.

9.3 Resistance of vertical walls

9.3.1 Design of individual unstiffened plates

(1) Unstiffened plates should be designed for bending as a two-dimensional plate under the actions from the stored liquid, the pressure above the liquid, stresses resulting from diaphragm action, and local bending action from attachments or piping.

9.3.2 Design of individual stiffened plates

(1) Corrugated or trapezoidal sheeting that spans in the horizontal direction should be designed for global bending under the actions from the stored liquid, the pressure above the liquid, stresses resulting from diaphragm action, and local bending action from attachments or piping.

(2) Effective bending properties and bending resistance of stiffened plates should be derived in accordance with EN 1993-1-3.

(3) The in-plane shear stiffness and shear resistance may be determined as analogous to that of the plane plate if the sheeting is continuously connected along all its boundaries to the adjacent members.

> **NOTE:** If the connection is on only parts of the vertical boundary (e.g. connection only in the troughs of the corrugation or trapezoidal sheeting), the stresses can increase dramatically and the stiffness can decrease dramatically. It is assumed that such constructions will not be used because of requirements of [AC₁⟩ liquid tightness ⟨AC₁] .

9.3.3 Global bending from direct action of the stored liquid and the pressure above the liquid

(1)   Horizontal bending resulting from the normal pressure on the wall should be considered. The loads should be supported by either one-way or two-way bending action.

9.3.4 Membrane stresses from diaphragm action

(1)   The design should take account of membrane tension stresses that develop in the walls as a result of hydrostatic pressures on opposing walls normal to the wall being considered.

(2)   The design should also take account of membrane compression stresses that can develop as a result of wind acting on other walls that are orthogonal to the wall being considered.

9.3.5 Local bending action from attachments or piping

(1)   Local bending action from attachments or piping should be avoided as far as possible. However, if this is not possible, a check should be made on the local stresses and deformations near the attachment.

9.4   Serviceability limit states

(1)   The serviceability limit states for walls of rectangular steel tanks should be taken as follows:

  – deformations or deflections which adversely affect the effective use of the structure

  – deformations, deflections and vibrations which 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.

10   Requirements on fabrication, execution and erection with relation to design

(1)   The tank should be fabricated and erected according to EN 14015 or EN 14620 and executed according to EN 1090, as appropriate.

11 Simplified design

11.1 General

(1)   The simplified analysis of this section may be applied where all the following conditions are satisfied:

- the tank structure is of the form shown in figure 11.1;

- the only internal actions are liquid pressure and gas pressure above the liquid surface;

- maximum design liquid level not higher than the top of the cylindrical shell;

- the following loadings can all be neglected: thermally induced loads, seismic loadings, loads resulting from uneven settlement or connections and emergency loadings;

- no course is constructed with a thickness less than that of the course above it, except for the zone adjacent to the eaves ring;

- the design value of the circumferential stress in the tank shell is less than 435 N/mm^2;

- for a spherical roof, the radius of curvature is between 0,8 and 1,5 times the diameter of the tank;

- for a conical roof, the slope of the roof is between 1 in 5 and 1 in 3 if the roof is only supported from the shell (no internal support);

- the design gradient of the tank bottom is not greater than 1:100;

- the bottom is fully supported or supported by closely spaced parallel girders;

- the characteristic internal pressure is not below −8,5 mbar and not greater than 60 mbar;

- the number of load cycles is such that there is no risk of fatigue failure.

(2)   The design yield stress throughout this chapter should be taken as:

$$f_{y,d} = f_y / \gamma_{M0}$$ (11.1)

where:

f_y is the characteristic yield strength of the steel;

γ_{M0} according to section 2.9.2.2
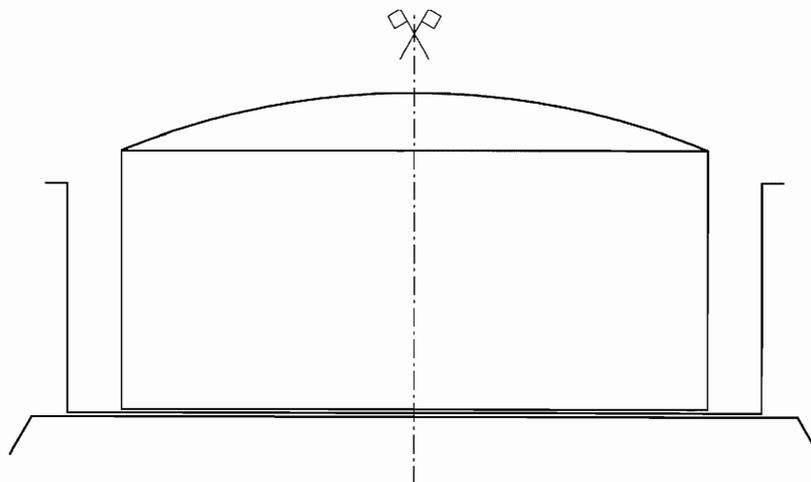
Figure 11.1: Tank structure with catch basin, where simplified tank design is applicable

11.2 Fixed roof design

11.2.1 Unstiffened roof shell butt welded or with double lap weld

(1)   Provided that the maximum local value of the distributed design load is used in (3) and (5) to represent the distributed pressure on the roof, possible non-uniformity of the distributed load need not be considered.

(2)   Where a concentrated load is applied, a separate assessment should be made in accordance with section 7.

(3)   The strength of the roof under the design internal pressure $p_{o,Ed}$ should be verified using:

- for spherical roofs $\qquad \dfrac{p_{0,Ed} R_s}{2t} \quad \leq \; j f_{y,d}$ $\qquad\qquad$ ... (11.2)

- for conical roofs $\qquad \dfrac{p_{0,Ed} R_c}{t} \quad \leq \; j f_{y,d}$ $\qquad\qquad$ ... (11.3)

in which:

$\qquad R_c = r / \sin\alpha \qquad\qquad$ for a conical roof

where:

$\quad j \qquad$ is the joint efficiency factor;

$\quad p_{o,Ed} \quad$ is the radial outward component of the uniformly distributed design load on the roof (i.e. the characteristic value multiplied by the partial factor according to section 2.9.2.1);

$\quad r \qquad$ is the radius of the tank cylindrical shell wall;

$\quad R_c \qquad$ is the radius of curvature for the conical roof;

$\quad R_s \qquad$ is the radius of curvature of the spherical roof;

$\quad t \qquad$ is the roof plate thickness;

$\quad \alpha \qquad$ is the slope of the conical roof to the horizontal.

40

(4)   The joint efficiency factor should be taken as:

j =   1,00 for butt welds;

j =   0,50 for lapped joints with fillet welds on both sides.

(5)   The stability of a spherical roof under the design external pressure $p_{i,Ed}$ should be verified using:

$$\boxed{AC_1}\ p_{i,Ed} \leq 0,05 \left\{ 1,25 E \left(\frac{t}{R_0} \right)^2 \right\} \boxed{AC_1} \qquad \dots (11.4)$$

in which:

R_0 = R_s

where:

$p_{i,Ed}$ is the radial inward component of the uniformly distributed design load on the roof (i.e. the characteristic value multiplied by the partial factor according to section 2.9.2.1).

(6)   The stability of a conical roof under the design external pressure $p_{i,Ed}$ should be verified according to the provisions of section 7.3 of EN 1993-4-1 $\boxed{AC_1}$ *text deleted* $\boxed{AC_1}$.

## 11.2.2 Self supporting roof with roof structure

(1)   The specified thickness of all roof plating should be not less than 3mm for stainless steels and not less than 5mm for other steels.

(2)   The roof structure should either be braced (see 11.2.4) or structurally connected to the roof plating.

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

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

(5)   Provided that the diameter of the tank is less than 60m and the distributed load does not deviate strongly from symmetry about the tank axis, the procedure described in (6) to (10) may be used for spherical roofs.
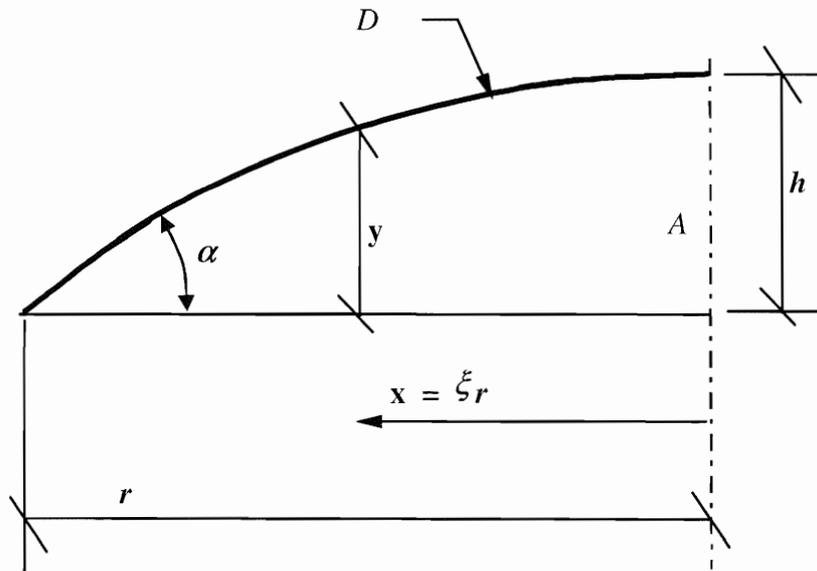
D= roof profile; A= tank axis

Figure 11.2: Tank spherical roof coordinates

(6) For spherical roofs 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 $p_{v,Ed}$ acting either upwards or downwards, with $p_{v,Ed}$ taken as negative if it acts upwards. The total design vertical force per rafter should be taken as:

$$P_{Ed} = \beta r^2 p_{v,Ed} \qquad \qquad ...(11.5)$$

in which:

$$\beta = \pi/n$$

where:

    n is the number of rafters;

    r is the radius of the tank;

    $p_{v,Ed}$ is the maximum vertical component of the design distributed load (see annex A) including the dead weight of the supporting structure (downward positive);

    P_{Ed} is the total design vertical force per rafter.

(7) The normal force N_{Ed} and bending moment M_{Ed} in each rafter for design according to EN 1993-1-1 may be obtained from:

$$N_{Ed} = 0,375 \frac{r}{h} P_{Ed} \qquad \qquad ...(11.6)$$

$$M_{Ed} = \frac{1}{3}\left(\frac{r}{1-\varepsilon}\right)\left\{1-\left(\frac{x}{r}\right)^3 - 1,10\left(\frac{y}{h}\right)\right\} P_{Ed} \qquad \qquad ...(11.7)$$

provided that the following conditions are met:

$$\boxed{\text{AC}_1}\; p_{v,Ed} \leq 1,2 \text{ kN/m}^2 \;\boxed{\text{AC}_1} \qquad \qquad ...(11.8)$$

$$\boxed{\text{AC}_1} \; I_y \geq \frac{N_{Ed} r^2}{\pi^2 E} \; \boxed{\text{AC}_1} \qquad\qquad\qquad \text{... (11.9)}$$

$$b_K \geq 2 h_K \qquad\qquad\qquad \text{... (11.10)}$$

$$A_1 \geq A_2 \qquad\qquad\qquad \text{... (11.11)}$$

$$h_K^2 \left(\frac{A_1 A_2}{A_1 + A_2} \right) \geq \frac{I_y}{2\beta} \qquad\qquad\qquad \text{... (11.12)}$$

in which:

$$\varepsilon = N_{Ed} \frac{(0,6r)^2}{\pi^2 E I_y} \qquad\qquad\qquad \text{... (11.13)}$$

where:

h is the rise of the tank roof, see figure 11.2;

x is the radial distance from the centreline of the tank, see figure 11.2;

y is the vertical height of the roof at coordinate x, see figure 11.2;

b_K is the flange width of the centre ring, see figure 11.3;

h_K is the vertical distance between the flanges of the centre ring, see figure 11.3;

A_1 is the area of the top flange of the centre ring, see figure 11.3;

A_2 is the area of the bottom flange of the centre ring, see figure 11.3;

I_y is the second moment of area of the rafter about the horizontal axis.

(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 $0{,}5r$ from the tank axis may be used in (7).

(9) Provided that the conditions given in (7) are satisfied, the design of the centre ring may be verified by checking only its lower chord according to (10).

(10) Provided that there are at least 10 uniformly spaced rafters, the design value of the member force $N_{r,Ed}$ and bending moment $M_{r,Ed}$ for the central ring may be calculated using:

$$N_{r,Ed} = \frac{N_{2,Ed}}{2\beta} \qquad\qquad\qquad \text{... (11.14)}$$

$$M_{r,Ed} = \frac{r_k \beta N_{2,Ed}}{2\left(3 + \beta^2\right)} \qquad\qquad\qquad \text{... (11.15)}$$

in which:

$$N_{2,Ed} = \frac{N_{Ed} e_o}{h_K} + \frac{M_{Ed}}{h_K} \qquad\qquad\qquad \text{... (11.16)}$$

where:

$N_{2.Ed}$ 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, see figure 11.3;

r_k is   the radius of the neutral axis of the centre ring, see figure 11.3.
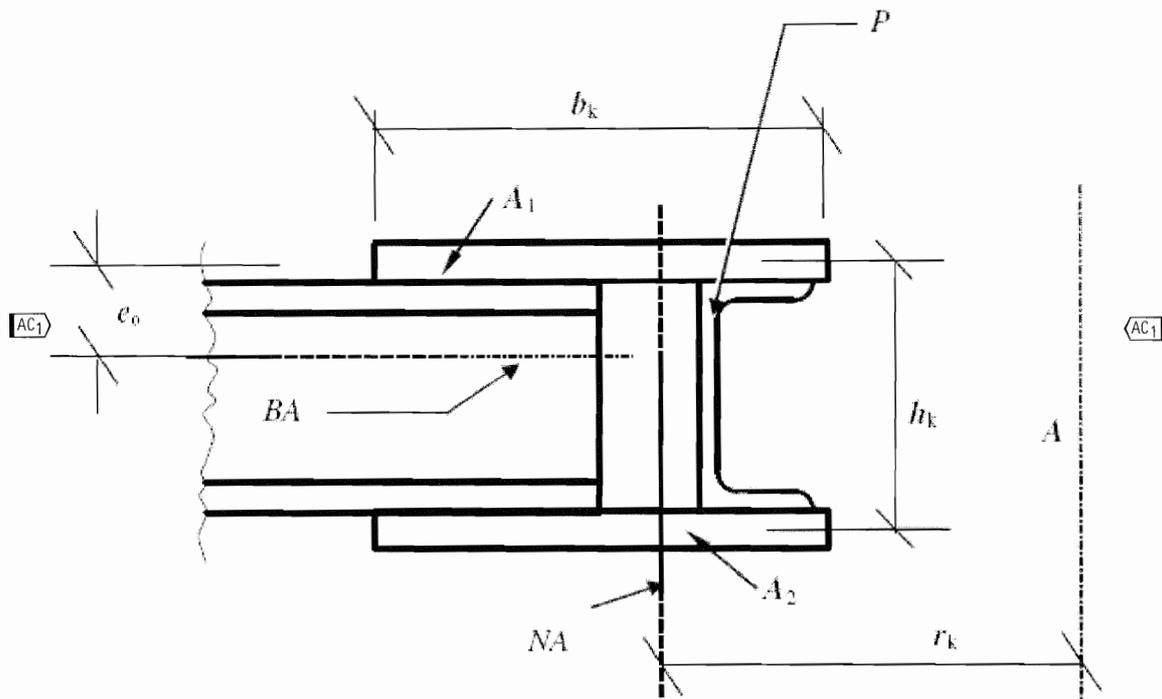
P= profile section separating flanges; *BA*= beam axis; *A*= tank axis; *NA*= neutral axis of A_1 and A_2 for bending in the plane of the plates

Figure 11.3: Roof centre ring

11.2.3 Column supported roof

(1)   The specified thickness of all roof plating should be not less than 3mm for stainless steels and not less than 5mm for other steels.

(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.

11.2.4 Bracing

(1)   If the roof plates are not connected to the rafters, bracing should be used.

(2)   For roofs exceeding 15 m diameter, at least two bays of bracing should be provided (i.e. two pairs of adjacent rafters connected by truss members). The sets of braced bays should be spaced evenly around the tank circumference.

(3)   For braced roofs with diameter between 15 m and 25 m, an additional circumferential ring should be provided. For braced roofs with diameter over 25 m, two additional circumferential rings should be provided.

(4)   The bracing should be designed for a stabilising force equal to 1% of the sum of the normal forces in the stabilised members.

11.2.5 Edge ring at the shell to roof junction (eaves junction)

(1)   The force in the effective edge ring (area where the roof is connected to the shell) should be verified using:

$$\frac{N_{Ed}}{A_{eff}} \leq f_{y,d} \qquad \qquad ...(11.17)$$

in which:

$$N_{Ed} = \frac{p_{v,Ed}\, r^2}{2 \tan \alpha} \qquad \qquad ...(11.18)$$

where:

A_{eff} is the effective area of the edge ring indicated in figure 11.4;

α is the slope of the roof to the horizontal at the junction;

$p_{v,Ed}$ is the maximum vertical component of the design distributed load including the dead weight of the supporting structure (downward positive).

(2)   Where the separation between adjacent rafters at their points of connection to the edge ring does not exceed 3,25m, the stability of the edge ring need not be verified.

(3)   Where the design distributed load $p_{v,Ed}$ acts upwards, the bending moments in the edge ring may be ignored.

(4)   Where the separation between adjacent rafters at their points of connection to the edge ring does not exceed 3,25m, and the design distributed load $p_{v,Ed}$ acts downwards, the bending moments in the edge ring may be ignored.

(5)   Where the separation between adjacent rafters at their points of connection to the edge ring exceeds 3,25m, the bending moments in the edge 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 inside of the ring) should be evaluated using the following expressions.
At the connection of the rafter:

$$M_{s,Ed} = -\left(\frac{p_{v,Ed}\, r^3}{2 \tan \alpha} \right)\left(1 - \frac{\beta}{\tan \beta} \right) \qquad \qquad ...(11.19)$$

At half span between the rafters:

$$M_{F,Ed} = -\left(\frac{p_{v,Ed}\, r^3}{2 \tan \alpha} \right)\left(\frac{\beta}{\sin \beta} - 1 \right) \qquad \qquad ...(11.20)$$

NOTE: Where $p_{v,Ed}$ acts in the upward direction, it is taken as negative, causing a change of sign in all the normal forces and bending moments.
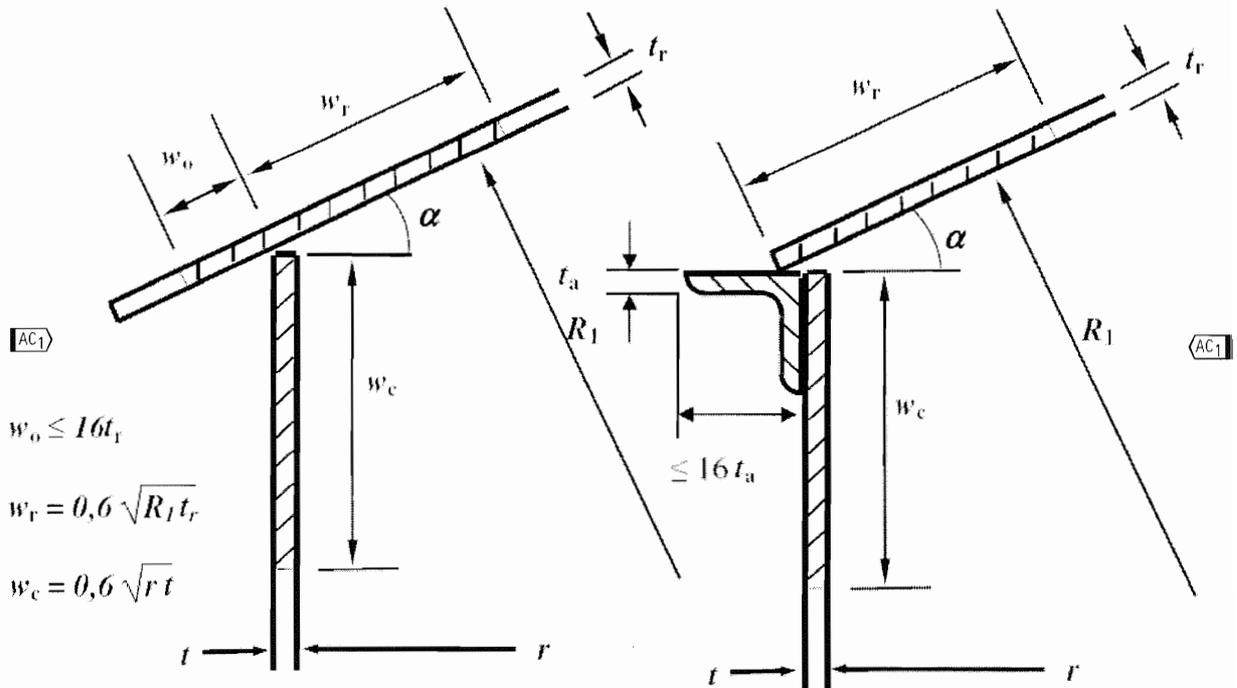
Figure 11.4: Edge ring at the shell to roof junction

In the figure:

$w_0 \leq 16t_r$

$w_r = 0{,}6\sqrt{R_1 t_r}$

$w_c = 0{,}6\sqrt{r\,t}$

$\leq 16\,t_a$

11.3 Shell design

11.3.1 Shell plates

(1) The circumferential normal stress due to liquid loads and internal pressure should be verified in each shell course using:

$$[\gamma_F \rho g H_{red} + p_{Ed}]\left(\frac{r}{t}\right) \leq f_{y,d} \qquad \ldots (11.21)$$

where the value of H_{red} for the jth course, denoted by $H_{red,j}$, is determined according to its relationship with the value for the course below it, which is the $(j\text{-}1)$th course:

$$H_{red,j} = H_j - \Delta H \qquad \text{if} \quad \frac{H_{red,j-1}}{f_{yd,j-1}} \geq \frac{H_{red,j}}{f_{yd,j}} \qquad \ldots (11.22)$$

$$H_{red,j} = H_j \qquad \text{if} \quad \frac{H_{red,j-1}}{f_{yd,j-1}} < \frac{H_{red,j}}{f_{yd,j}} \qquad \ldots (11.23)$$

in which:

$\Delta H \quad = 0{,}30$ metres

where:

$\rho \quad$ is the density of the contained liquid;

$g \quad$ is the acceleration due to gravity;

46

H_j is the vertical distance from the bottom of the jth course to the liquid level;

p_{Ed} is the design value of the pressure above the liquid level (i.e. the characteristic value multiplied by the partial factor according to ⟨AC₁⟩ 2.9.2.1 ⟨AC₁⟩).

11.3.2 Stiffening rings

(1)   Fixed roof tanks with roof structures may be considered to be adequately stiffened at the top of the shell by the roof structure. A primary ring need not be used.

(2)   Open top tanks should be provided with a primary ring which is located at or near the top of the uppermost course.

(3)   If the lower edge of the shell is effectively anchored to resist vertical displacements the primary stiffening ring may be designed by satisfying both the strength and the stiffness requirement given in clauses (12) to (14) of section 5.3.2.5 of EN 1993-4-1.

(4)   If the lower edge of the shell is not effectively anchored to resist vertical displacements the buckling assessment should be carried out using EN 1993-1-6.

(5)   When stiffening rings are located more than 600 mm below the top of the shell, the tank should be provided with a top curb angle with the following size:

    −   60×60×5 where the top shell course has a thickness less than 6 mm;

    −   80×80×6 where the top shell course has a thickness of 6 mm or more.

For either angle section, the horizontal leg should be not further than 25 mm from the top edge of the shell.

(6)   The requirement for a secondary ring to prevent local buckling of the shell should be investigated using the following procedure. The height over which buckling of the unstiffened shell can occur (measured from the top of the shell or the primary wind girder downwards) should be taken from:

$$H_E = \sum h \left(\frac{t_{min}}{t} \right)^{2.5} \qquad\qquad \text{... (11.24)}$$

where:

h is the height of each course in turn below the edge ring or the primary wind girder;

t is the thickness of each course in turn;

t_{min} is the thickness of the thinnest course.

(7)   The height that may be taken to be stable without a secondary ring should be taken from:

$$H_P = 0{,}46 \left(\frac{E}{p_{Ed}} \right) \left(\frac{t_{min}}{r} \right)^{2.5} rK \qquad\qquad \text{... (11.25)}$$

in which:

$K = 1$ if the axial stress $\sigma_{x,Ed}$ is tensile   ... (11.26)

$$K = \left\{ 1 - \left[2{,}67 \left(\frac{\sigma_{x,Ed}}{E} \right) \left(\frac{r}{t} \right) \left(1 + \frac{1}{54} \left(\frac{r}{t} \right)^{0{,}72} \right)^{1{,}25} \right]^{0{,}8} \right\} \quad \text{if the axial stress is compressive.. (11.27)}$$

where p_{Ed} is the maximum design value of the inward component of the pressure on the shell wall (pressure on the outside, negative pressure on the inside) and (r/t) is taken at the same location as the design value $\sigma_{x,Ed}$ of the compressive axial membrane stress.

> **NOTE:** The above formulas can sometimes be very conservative (especially in the case of very short courses). The provisions of EN 1993-1-6 may always be used to provide a more economic design.

(8)   The non-uniform distribution of pressure $q_{w,Ed}$ resulting from external wind loading on cylinders (see figure 11.5) may, for the purpose of tank buckling design, be substituted by an equivalent uniform external pressure:

$$q_{eq,Ed} = k_w \, q_{w,max,Ed} \qquad \qquad \text{... (11.28)}$$

where $q_{w,max,Ed}$ is the maximum wind pressure, and k_w should be found as follows:

$$k_w = 1/C_w \qquad \qquad \text{... (11.29)}$$

with C_w according to clause (8) of section 5.3.2.5 of EN 1993-4-1.

(9)   The pressure p_{Ed} to be introduced into 11.25 follows from:

$$p_{Ed} = q_{eq,Ed} + q_{s,Ed} \qquad \qquad \text{... (11.30)}$$

where $q_{s,Ed}$ is the internal suction caused by venting, internal partial vacuum or other phenomena.

a) wind pressure distribution around shell circumference

b) equivalent axisymmetric pressure distribution
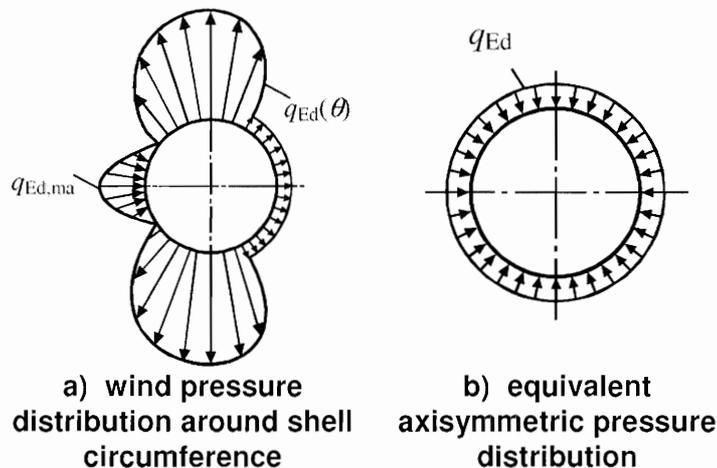
Figure 11.5: Transformation of typical wind external pressure load distribution

(10)   The procedure set out in (7) should not be used where the axial stress is compressive unless both of the following conditions are met:

$$\frac{r}{t} \geq 200 \qquad \qquad \text{... (11.31)}$$

$$f_y \geq 1,15 E \left(\frac{r}{\ell}\right)\left(\frac{t}{r}\right)^{1.5} \qquad \qquad \text{... (11.32)}$$

where:

ℓ is   the height of the buckle. This is given by H_E or the distance between the adjacent ring stiffeners whichever is less.

(11)   If $H_E \leq H_P$, a secondary ring need not be used.

(12)   If $H_E > H_P$, the height H_E should be subdivided by stiffening rings equally spaced at separations H_P or less to prevent buckling of the shell wall. If more than one stiffening ring is necessary, the value of K may be calculated separately for each bay between stiffening rings, to give different distances H_P between stiffening rings according to (7).

(13)   If the thickness of the course to which a lower ring is attached is greater than the minimum plate thickness t_{min}, an adjustment should be made as follows. The distance $H_{lower,adj}$ at which a lower ring should be placed below the edge ring or primary ring should be evaluated instead as using:

$$H_{lower,adj} = H_{tmin} + (H_{lower} - H_{tmin})\left(\frac{t}{t_{min}}\right)^{2.5} \qquad \qquad \text{... (11.33)}$$

where:

H_{lower} is   the distance from the edge ring or the primary ring to the secondary ring position to be adjusted;

H_{tmin} is   the distance from the edge ring or the primary ring to the lower boundary of the shell courses with thickness t_{min}.

(14)   Secondary rings should not be located within 150mm of a circumferential tank seam.

(15)   Unless a more detailed assessment is carried out using EN 1993-1-6 secondary rings should satisfy the following stiffness requirement

$$I_{R.j} \geq 2\frac{N_{Rj,Ed}r^2}{Em_B^2} \qquad \qquad \text{(11.34)}$$

with

$$N_{R.j.Ed} = \frac{p_{j,Ed}r(a_{j+1}+a_j)}{2} \qquad \qquad \text{(11.35)}$$

$$m_B^* = 1,79 \left\{\frac{r}{H\left[\dfrac{r^2\min(a_j t_j)}{\max(I_{R.j})}\right]^{1/4}}\right\}^{1/2} \qquad \qquad \text{(11.36)}$$

m_B =   next smaller integer to m_B^*

$I_{R.j}$ second moment of area of secondary ring j

$\max I_{R.j}$ maximum value of $I_{R.j}$ for all secondary rings

H height of the primary ring or roof to shell junction above the bottom edge

a_j distance from secondary ring j to the next secondary ring below or to the bottom edge if there is no secondary ring below

a_{j+1} distance from secondary ring j to the next secondary ring above or to the primary ring or roof to shell junction if there is no secondary ring above

t_j mean value of the shell thickness along the distance a_j

$\min(a_j t_j)$ minimum value of $a_j t_j$ along the height H

r radius of the tank shell

$p_{j.Ed}$ negative design pressure at the secondary ring j

11.3.3 Openings

(1)   Openings and mountings should be designed according to 5.4.6.

11.4 Bottom design

(1)   The design of the bottom plate should take corrosion into account.

(2)   Bottom plates should be lap welded or butt welded. For welding details see EN 14015 or EN 14620, as appropriate.

(3)   The specified thickness of the bottom plates should not be less than specified in table 11.1 excluding corrosion allowance. Larger values should be used if required to resist uplift due to the internal negative pressure, unless a minimum guaranteed residual liquid level is used to assist in resisting this uplift.

Table 11.1: Minimum nominal bottom plate thickness

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

(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.

(5)   Bottoms for tanks greater than 12,5m diameter should have a base ring (in the form of an annular plate) that satisfies the strength and toughness requirements on the shell course to which they are attached. This base ring should have a minimum nominal thickness t_a excluding corrosion allowance obtained from:

$$t_a = t_s/3 + 3\text{mm} \qquad \text{but not less than 6mm} \qquad \qquad ... (11.37)$$

where t_s is the thickness of the attached shell course.

NOTE 1: This minimum thickness of bottom plate may lead to the formation of a plastic hinge in the annular plate, avoiding alternating plasticity in the weld detail at the bottom of the shell wall. However, it should be noted that this minimum plate thickness may also lead to uplift of the outer edge of the annular plate, with consequent potential for corrosion.

NOTE 2: Where axial forces develop in the tank shell, the annular plate must be designed to distribute these axial forces into the foundation.

(6)   The inner part of the base ring annular plate should not have an exposed width w less than the limiting value w_a, obtained from:

$$w_a = 1,5 \left[\frac{f_y t_a^2}{\rho g H} \right]^{1/2} \qquad \text{but not less than 500mm} \qquad (11.38)$$

where:

H is the maximum design liquid height.

w_a is the minimum exposed width (distance from the inner edge of the annular base plate to the inner edge of the shell plate).

t_a is the thickness of the annular plate, taking account of the corrosion allowance.

ρ is the density of the contained liquid.

g is the acceleration due to gravity.

(7)   The radial seams connecting annular plates to each other should be full penetration butt welded. For welding details, see EN14015 or EN14620, as appropriate.

(8)   The distance from the outer edge of the shell plate to the outer edge of the bottom plates or base ring annular plate should not be less than 50mm.

(9)   The attachment of the lowest course of the shell plate to the annular plates or bottom sketch plates should be continuous fillet welds on both sides of the shell plate.

(10)   The throat thickness for each fillet weld should be greater than or equal to the thickness of the annular plate or of the sketch plate, except that they should not exceed 10mm and where the shell plate thickness is less than the sketch plate or annular plate thickness, they should not exceed the appropriate value given in table 11.2.

Table 11.2: Fillet weld throat thickness if shell plate is thinner than sketch plate or annular plate

Shell plate thickness, t	Fillet weld throat thickness
$t < 5$ mm	2,0 mm
$t = 5$ mm	4,5 mm
$t > 5$ mm	6,0 mm

11.5 Anchorage design

(1)   Tank anchorage should be provided for fixed roof tanks, if any of the following conditions can 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;

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.

c)   Uplift of an empty tank due to wind loading counteracted by the effective corroded weight of roof, shell and permanent attachments;

d)   Uplift of an empty tank due to external liquid caused by flooding. In such cases it is necessary to consider the effects upon the tank bottom, tank shell etc. as well as the anchorage design.

For this check, the uplift forces due to the wind load may be calculated using the assumption that the tank shell has a rigid cross section (beam theory). This assumption implies that local uplift can occur. In cases where no local uplift can be allowed, a more sophisticated analysis is required.

(2)   Anchorage points should be spaced evenly around the circumference of the tank, insofar as this is possible.

(3)   The design of the holding down bolts or straps should meet the requirements of EN 1993-1-1. The minimum cross-sectional area for the holding down bolts or straps should be 500mm^2. If corrosion is anticipated, a minimum corrosion allowance of 1mm should be added.

(4)   The anchorage should be principally attached to the shell wall. It should not be attached to the bottom plate alone.

(5)   The design of the anchorage should accommodate movements of the tank due to thermal changes and hydrostatic pressure and minimise any stresses induced in the shell.

(6)   The design of the shell for local anchorage forces and bending moments resulting from the anchorage should meet the requirements of 5.4.6 and 5.4.7 of EN 1993-4-1.

(7)   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 pretensioned, the maximum uplift forces in them under wind load will be reduced, so that the calculation described in (1) will be applicable. In addition, a reduction will occur in the stresses induced by restraint of radial movements due to thermal changes and hydrostatic pressure.

Annex A [normative]

Actions on tanks

A.1 General

(1)   The design should take account of the characteristic values of the actions listed in A.2.1 to A.2.14.

(2)   The partial factors on actions according to 2.9.2.1 and the action combination rules according to 2.10 should be applied to these characteristic values.

A.2 Actions

A.2.1 Liquid induced loads

(1)   During operation, the load due to the contents should be the weight of the *product to be stored from maximum design liquid level* to empty.

(2)   During test, the load due to the contents should be the weight of the *test medium from maximum test liquid level* to empty.

A.2.2 Internal pressure loads

(1)   During operation, the internal pressure load should be the load due to the specified minimum and maximum values of the internal pressure.

(2)   During test, the internal pressure load should be the load due to the specified minimum and maximum values of the test internal pressure.

A.2.3 Thermally induced loads

(1)   Stresses resulting from restraint of thermal expansion may be ignored if the number of load cycles due to thermal expansion is such that there is no risk of fatigue failure or cyclic plastic failure.

A.2.4 Dead loads

(1)   The dead loads on the tank should be considered as those resulting from the weight of all component parts of the tank and all components permanently attached to the tank.

(2)   Numerical values should be taken from EN 1991-1-1.

A.2.5 Insulation loads

(1)   The insulation loads should be those resulting from the weight of the insulation.

(2)   Numerical values should be taken from EN 1991-1-1.

A.2.6 Distributed live load

(1)   The distributed live load should be taken from EN 1991-1-1 unless otherwise specified.

A.2.7 Concentrated live load

(1)   The concentrated live load should be taken from EN 1991-1-1 unless otherwise specified.

A.2.8 Snow

(1)   The loads should be taken from EN 1991-1-3.

A.2.9 Wind

(1)   The loads should be taken from EN 1991-1-4.

(2)   In addition, the following pressure coefficients may be used for circular cylindrical tanks, see figure A.1:

  a)   internal pressure of open top tanks and open top catch basin: $c_p = -0,6$.

  b)   internal pressure of vented tanks with small openings: $c_p = -0,4$.

  c)   where there is a catch basin, the external pressure on the tank shell may be assumed to reduce linearly with height.

(3)   Due to their temporary character, reduced wind loads may be used for erection situations according to EN 1991-1-4.

A.2.10 Suction due to inadequate venting

(1)   The loads should be taken from EN 1991-1-4.

A.2.11 Seismic loadings

(1)   The loads should be taken from EN 1998-4, which also sets out the requirements for seismic design.

### A.2.12 Loads resulting from connections

(1)   Loads resulting from pipes, valves and other items connected to the tank and loads resulting from settlement of independent item supports relative to the tank foundation should be taken into account. Pipework should be designed to minimise loadings applied to the tank.

### A.2.13 Loads resulting from uneven settlement

(1)   Settlement loads should be taken into account where uneven settlement can be expected during the lifetime of the tank.

A.2.14 Emergency loadings

(1)   The loads should be specified for the specific situation and can include loadings from events such as external blast, impact, adjacent external fire, explosion, leakage of inner tank, roll over, overfill of inner tank.
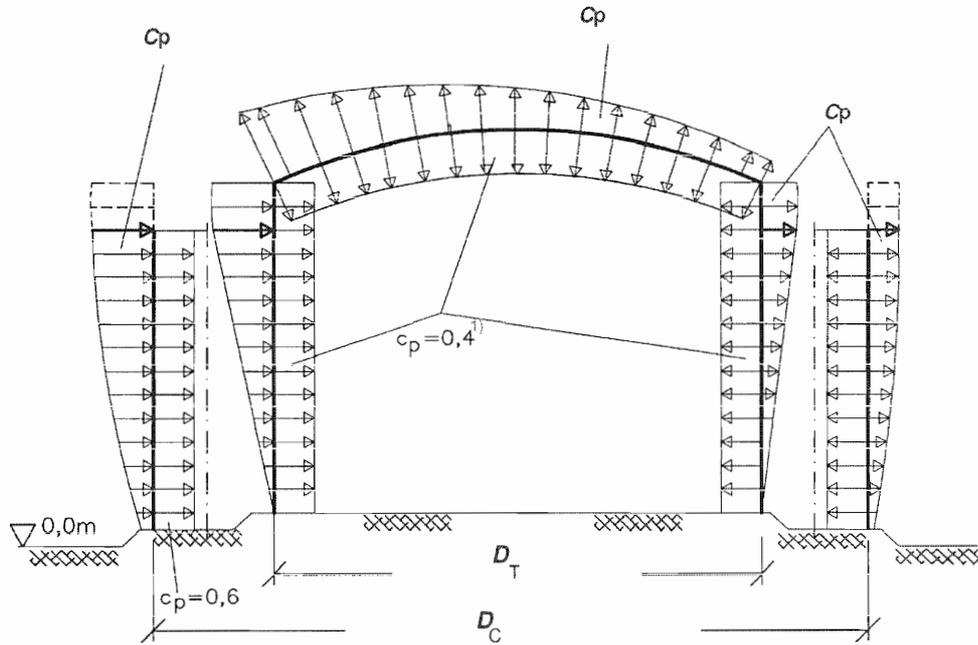
a) Tank with catch basin
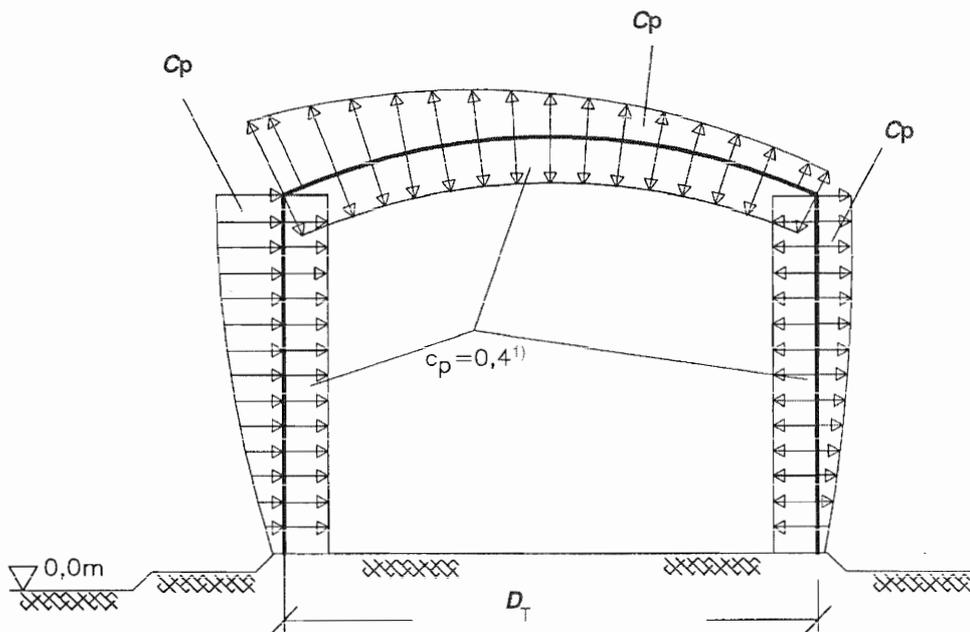
b) Tank without catch basin

D_T= Diameter of tank; D_c= Diameter of catch-basin;

1) [AC₁⟩ c_p = 0,4 applies only for the vented tank; where no numerical values are given with c_p ⟨AC₁]
they have to be obtained from EN 1991-1-4.

Figure A.1: Pressure coefficients for wind loading on a circular cylindrical tank