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Will supersede EN 1993-1-7:2007

English Version

Eurocode 3 - Design of steel structures - Part 1-7: Plate assemblies with elements under transverse loads

Eurocode 3 - Calcul des structures en acier - Partie 1-7 : Structures en plaques avec éléments sous charges transversales

Eurocode 3 - Bemessung und Konstruktion von Stahlbauten - Teil 1-7 : Aus Blechen zusammengesetzte Bauteile unter Querbelastung

This draft European Standard is submitted to CEN members for enquiry. It has been drawn up by the Technical Committee CEN/TC 250.

If this draft becomes a European Standard, 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.

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EUROPEAN COMMITTEE FOR STANDARDIZATION
COMITÉ EUROPÉEN DE NORMALISATION
EUROPÄISCHES KOMITEE FÜR NORMUNG

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European foreword

This document (prEN 1993-1-7:2023), has been prepared by Technical Committee CEN/TC250 “Structural Eurocodes”, the secretariat of which is held by BSI. CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

This document will supersede EN 1993-1-7:2007 and its amendments.

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

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

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

0 Introduction

0.1 Introduction to the Eurocodes

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

— EN 1990 Eurocode: Basis of structural and geotechnical design

— EN 1991 Eurocode 1: Actions on structures

— EN 1992 Eurocode 2: Design of concrete structures

— EN 1993 Eurocode 3: Design of steel structures

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

— EN 1995 Eurocode 5: Design of timber structures

— EN 1996 Eurocode 6: Design of masonry structures

— EN 1997 Eurocode 7: Geotechnical design

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

— EN 1999 Eurocode 9: Design of aluminium structures

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

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant authorities (in exercising their duties in accordance with national or international regulations), educators, software developers, and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

0.2 Introduction to EN 1993 (all parts)

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

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

EN 1993 is subdivided in various parts:

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

EN 1993-2, *Design of Steel Structures — Part 2: Steel bridges;*

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

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

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

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

EN 1993-7, *Design of steel structures — Part 7: Design of sandwich panels.*

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

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

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

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

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

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

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

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

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

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

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

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

EN 1993-1-11, *Design of Steel Structures — Part 1-11: Design of structures with tension components;*

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

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

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

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

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

0.3 Introduction to prEN 1993-1-7

prEN 1993-1-7 gives supplementary rules for plate assemblies with elements under transverse loads.

1 Under preparation.

2 Under preparation.

0.4 Verbal forms used in the Eurocodes

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

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

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

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

0.5 National Annex for prEN 1993-1-7

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

The national standard implementing prEN 1993-1-7 can have a National Annex containing all national choices to be used for the design of steel structures to be constructed in the relevant country.

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

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

National choice is allowed in prEN 1993-1-7 through the following clauses:

4.1(5)	4.6(2)	9.2.2.2(5)

National choice is allowed in prEN 1993-1-7 on the application of the following informative annexes:

Annex A	Annex B	Annex C

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

1 Scope

1.1 Scope of prEN 1993-1-7

(1) prEN 1993-1-7 provides rules for the structural design of assemblies of unstiffened and stiffened steel plates whose elements are under predominantly distributed transverse loads.

(2) prEN 1993-1-7 is applicable to containment structures such as silos, tanks, digesters and lock gates, where the external actions chiefly act transversely on their individual plates or panels. Where a plate or panel under bending is additionally subject to membrane forces that have a significant effect on the resistance, this document covers assessment of the resistance through its computational analysis procedures.

(3) prEN 1993-1-7 is applicable to structures with rectangular, trapezoidal or triangular component plate segments, each with one axis of symmetry.

(4) prEN 1993-1-7 does not apply to plates or panels where the dominant structural resistance requirement relates to membrane forces in the plates (for these, see EN 1993-1-5).

(5) prEN 1993-1-7 does not apply to plates or panels whose curvature (out of flatness) exceeds that defined in 1.1 (14). For such curved plates, see EN 1993-1-6.

(6) prEN 1993-1-7 does not apply to circular or annular plates. For such plates, see EN 1993-1-6.

(7) prEN 1993-1-7 does not apply to cold-formed sheeting. For such plates, see EN 1993-1-3.

(8) This document is only concerned with the requirements for design of plates and plate assemblies against the ultimate limit states of:

— plastic failure;

— cyclic plasticity;

— buckling;

— fatigue.

(9) Overall equilibrium of the structure (sliding, uplifting, or overturning) is not included in this document. Special considerations for specific applications are available in the relevant applications parts of EN 1993.

(10) The rules in this document refer to plate assemblies that are fabricated using unstiffened or stiffened plates or panels. The document is also applicable to the design of individual plates or panels that are predominantly subject to actions transverse to the plane of each plate. Both frictional actions on the plate surface and forces imposed by adjacent components of the plate assembly also induce in-plane actions in each plate.

(11) This document gives algebraic rules and guidance to account for bending with small membrane forces in the individual plates or panels. Where an unstiffened or stiffened plates or panels is subject to significant magnitudes of both bending and in-plane forces, the computational analysis procedures of this document apply.

(12) Where no application part defines a different range, this document applies to structures within the following limits:

— design metal temperatures within the range −50 °C to +100 °C;

— the geometry of individual plate segments is limited to rectangular, triangular and trapezoidal shapes with b/t greater than 20, or b_1/t greater than 20, as appropriate (see Figure 3.2);

— Single plate elements are treated as flat where the deviation from flatness e_0 meets the condition $e_0/t \leq 0,750$ (see Figure 9.1). Where this criterion is not met, it is appropriate to treat the plate as a shell panel (see EN 1993-1-6).

1.2 Assumptions

(1) Unless specifically stated, the provisions of EN 1990, EN 1991 (all parts) and EN 1993 (all parts) apply.

(2) The design methods given in prEN 1993-1-7 are applicable if:

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

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

(3) The provisions in this document apply to materials that satisfy the brittle fracture provisions given in EN 1993-1-4 and EN 1993-1-10.

(4) In this document, it is assumed that wind loading, seismic actions and bulk solids flow can, in general, be treated as quasi-static actions.

(5) Dynamic effects are treated in other relevant application parts of EN 1993 or EN 1998, including the consequences for fatigue. The stress resultants arising from dynamic behaviour are treated in this part as quasi-static.

2 Normative references

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

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. through 'should' clauses) and permissions (i.e. through 'may' clauses).

EN 1090-2, *Execution of steel structures and aluminium structures - Part 2: Technical requirements for steel structures*

EN 1090-4, *Execution of steel structures and aluminium structures - Part 4: Technical requirements for cold-formed structural steel elements and cold-formed structures for roof, ceiling, floor and wall applications*

EN 1990, *Basis of structural and geotechnical design*

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

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

ISO 8930, *General principles on reliability for structures - Vocabulary*

3 Terms, definitions and symbols

For the purposes of this document, the terms and definitions given in EN 1990, EN 1993-1-1, ISO 8930 and the following apply.

3.1 Terms and definitions

3.1.1 Structural forms and geometry

3.1.1.1
plate
structural element that, in general, has two large dimensions a and b and a uniform much smaller dimension t, and is shaped such that the two large dimensions lie in a single plane (flat plate)

Note 1 to entry: In this standard, the ratios a/t and b/t are required to exceed the value 20 (except as noted below). Where the boundary conditions and geometry are such that the plate only bends in a single direction, its treatment is not the principal role of this standard, but the provisions given here can be used.

Note 2 to entry: In special circumstances (e.g. the edge b_2 of a trapezoidal plate), the smaller dimension b can be less than $20t$.

Note 3 to entry: A plate for which the above restrictions apply is termed a thin plate.

Note 4 to entry: Typical generic forms of plate assemblies considered by this standard are illustrated in Figures 3.1 and 3.3. These are categorised as assemblies of plates of rectangular, trapezoidal or triangular shape, each plate element having at least one axis of symmetry.

3.1.1.2
plate assembly
structure that is assembled from flat plates which are joined together (see Figure 3.1) in such a way that the assembly has at least one axis of symmetry

Note 1to entry:The individual plates may be unstiffened or stiffened

Note 2 to entry: The coordinate system indicated in Figure 3.2 only serves to indicate directions. The origin can be chosen by the user to be at any suitable location.

Note 3 to entry: The dimensions a, b and c shown in Figure 3.1 relate to the complete plate assembly to give clarity to the usage in this standard for common orientations of plate. Where an individual plate is described elsewhere in the standard, the dimension a is the longer side and the shorter side is always b, even if in the global system it is defined as c.

3.1.1.3
plate geometry
geometries of individual plates that are defined as rectangular, trapezoidal or triangular

Note 1to entry:. Where the shape is rectangular, the larger side length is defined as the dimension a. Where the shape is other than rectangular, the side(s) parallel to the axis of symmetry are defined by the dimension a (see Figure 3.2). Trapezoidal and triangular plates are only covered by this standard where they have an axis of symmetry.

3.1.1.4
panel
flat plate which may be unstiffened or stiffened

Note 1to entry:. A panel can be regarded as an individual part of a plate assembly (see Figure 3.1). The term can also be used for stiffened plates with transverse and longitudinal stiffeners, which delimit sub-panels (see 3.1.1.11).

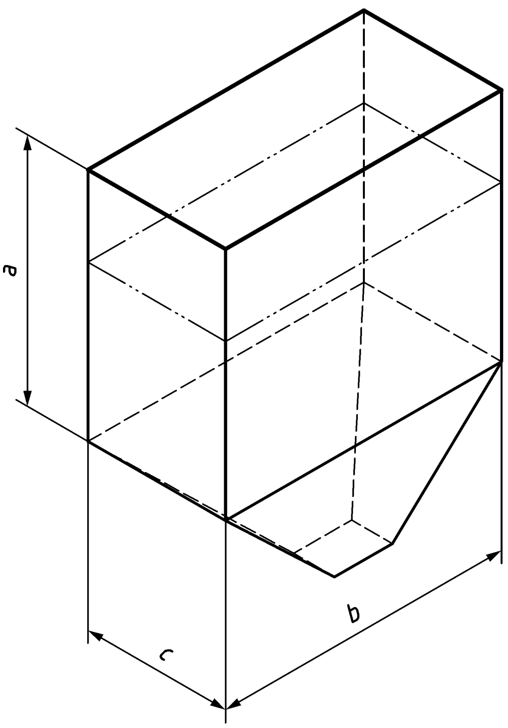


Figure 3.1 — Typical arrangement of a plate assembly, composed of individual panels, that is unstiffened or stiffened plates

3.1.1.5
aspect ratio
ratio of the shorter side length to the longer side length ($\psi = b/a \leq 1{,}0$) for a rectangular plate or panel

3.1.1.6
stiffener
flat plate or prismatic member attached to a panel for the purpose of increasing its bending resistance

Note 1 to entry: It can also be used to reinforce the member to support local loads.

3.1.1.7
longitudinal stiffener
stiffener on a rectangular panel in which the stiffener longitudinal axis is aligned with the longer dimension *a* of the panel (see Figure 6.8)

3.1.1.8
transverse stiffener
stiffener on a rectangular panel in which the stiffener longitudinal axis is aligned with the shorter dimension *b* of the panel (see Figure 6.8)

Note 1 to entry: The term "transverse stiffener" is commonly used in plates that are subject only to membrane forces to refer to stiffeners that are orthogonal to the direction of a main membrane force. By contrast, in this standard the terminology of 3.1.1.7 and 3.1.1.8 defines the longitudinal and transverse directions only in terms of the shape of the plate, since these plates are principally subject to bending in both directions.

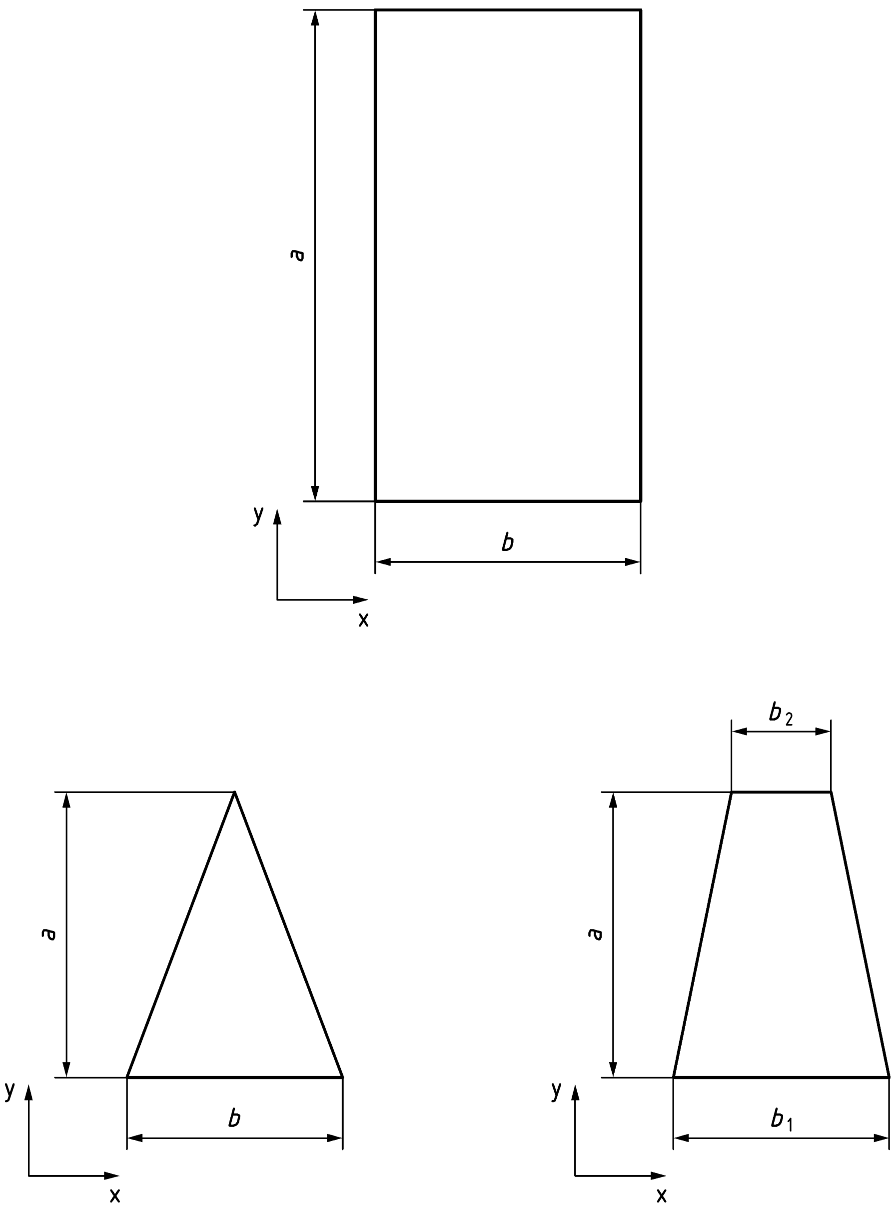


Figure 3.2 — Dimensions and local coordinate systems for rectangular, triangular and trapezoidal plates

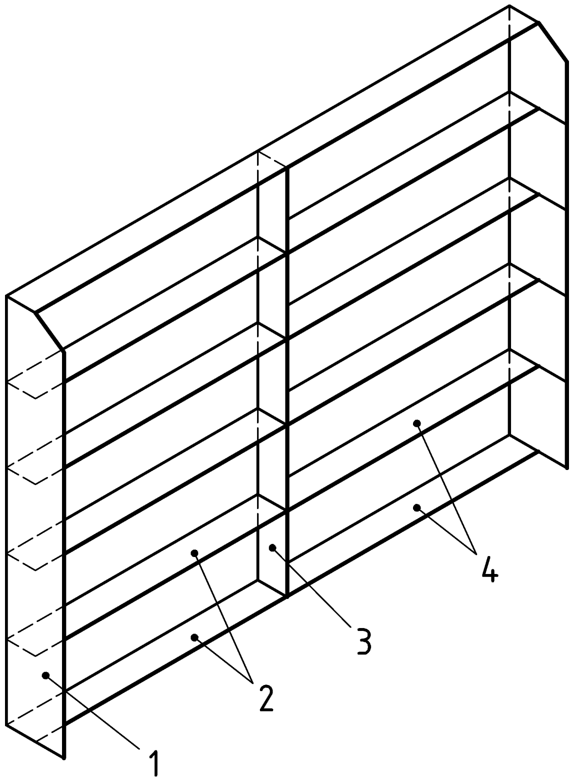
3.1.1.9
uni-directionally stiffened plate
rectangular plate that has parallel stiffeners attached to it with their longitudinal axis in one direction

Note 1 to entry: The direction can be longitudinal or transverse.

3.1.1.10
bi-directionally stiffened plate
rectangular plate that has two sets of parallel stiffeners in the two principal directions attached to it with their longitudinal axes in orthogonal directions

3.1.1.11
sub-panel
part of a stiffened plate that lies between stiffeners, and so is locally an unstiffened plate bounded by stiffeners

Note 1 to entry: The design of sub-panels is covered within the rules of this standard in 6.6 and Clauses 9 and 10, where stiffened plates are treated.



Key

1 transverse end stiffener
2 longitudinal stiffeners
3 transverse intermediate stiffener
4 sub-panels

Figure 3.3 — Example of a rectangular stiffened plate

3.1.2 Failure mechanisms

3.1.2.1
buckling
ultimate limit state where the stability of the structure is lost under compression and/or shear

3.1.2.2
cyclic plasticity
ultimate limit state in which repeated cycles of loading lead to repeated plastic straining

Note 1 to entry: Two distinct failure modes can arise: ratcheting and low-cycle fatigue.

3.1.2.3
high cycle fatigue
ultimate limit state where a high number of cycles of loading and unloading under nominally elastic stresses induce a fatigue crack

3.1.2.4
low cycle fatigue
ultimate limit state where repeated alternating cycles of plastic strain cause exhaustion of the plasticity of the material

3.1.2.5
plastic failure
ultimate limit state where the structure loses its ability to resist increased loading due to the development of excessive plastic deformations

3.1.2.6
ratcheting
progressive increase of plastic strains up to failure in the direction of the mean stress caused by unsymmetrical cycles of stress

3.1.2.7
tensile rupture
ultimate limit state where separation of the parts of a panel or the junctions between panels occurs due to tension

3.1.3 Actions

3.1.3.1
transverse load
pressure loading applied to the plate normal to its middle surface (perpendicular to both the dimensions a and b)

3.1.3.2
in-plane loading
forces applied parallel to or in the plane of the middle surface of a plate

Note 1 to entry: The forces can be applied through the connections between panels, or by frictional loads applied to the plate surface, or by temperature effects, or where large displacements cause some of the transverse loads acting on an individual panel to be carried by forces in its plane.

3.1.4 Terms for analysis treatments

3.1.4.1
computational analysis
use of analysis software (usually finite element) to produce a numerical analysis of the structure

Note 1 to entry: This can take different forms depending on the assumptions adopted in the numerical model (see 6.1).

3.1.4.2
global analysis
analysis that includes the complete structure, rather than individual structural parts treated separately

Note 1 to entry: This is usually a computational analysis.

3.1.4.3
membrane and simple bending analysis (MSBA)
analysis using simple statics of membrane forces and simple bending analysis treating each plate or panel as separate (see 6.1)

3.2 Symbols

For the purposes of this document, the symbols given in EN 1990 and EN 1993-1-1 and the following apply.

NOTE 1 Symbols and notations which are not listed below are explained in the text where they first appear.

Latin upper case letters

E Young's modulus of elasticity

F generalized action

F_{Ed} action set on a complete structure corresponding to a design situation (design values)

F_{Rd} calculated values of the action set at the maximum resistance condition of the structure (design values)

R resistance of the structure under the design values of loads in a specific load case

R_{cr} critical buckling resistance ratio (defined as a load factor on design loads using LBA analysis)

R_{k} characteristic reference resistance ratio (used with subscripts to identify the basis): defined as a load factor on design loads using the ratio (F_{Rk} / F_{Ed})

R_{pl} plastic reference resistance ratio (defined as a load factor on design loads using MNA analysis)

R_{plf} plastic failure resistance ratio (defined as a load factor on design loads using GMNA analysis)

R_{GNA} buckling resistance ratio determined in a GNA analysis

R_{GMNA} buckling resistance ratio determined in a GMNA analysis

R_{GMNIA} buckling resistance ratio determined in a GMNIA analysis (normally as R_{k})

Latin lower case letters

a length of a rectangular plate or panel (longer dimension), or length of a symmetrical triangular or trapezoidal plate or panel parallel to the axis of symmetry, see Figure 3.2

b width of a rectangular plate or panel (shorter dimension), or base of a symmetrical triangular plate or panel, see Figure 3.2

b_1 length of the longer side normal to the axis of symmetry in a symmetrical trapezoidal plate or panel , see Figure 3.2

b_2 length of the shorter side normal to the axis of symmetry in a symmetrical trapezoidal plate or panel, see Figure 3.2

e eccentricity of the equivalent axial force N_{Ed} in the plate and stiffener assembly relative to the centroid of the effective cross-section

f_y yield stress or 0,2% proof stress for material with a nonlinear stress-strain curve

t uniform thickness of a plate

x_e exclusion distance

NOTE 2 The dimension "*a*" is defined in different senses in common texts on plates, making no single notation universal. In this document the use of "*a*" as the longer side of a rectangular plate element is to provide consistency with other Eurocodes, notably with EN 1993-1-1. The notation for triangular and trapezoidal plates is only used in this document.

Greek upper case letters

Δ Mathematical operator indicating a change in a value

Greek lower case letters

ψ aspect ratio of a rectangular plate or panel ($b/a \leq 1,0$)

ε strain

ρ reduction factor for plate buckling

ν Poisson's ratio

γ_M partial factor for resistance

γ_{M0} partial factor for plastic resistance or material yielding

γ_{M1} partial factor for resistance to stability (buckling)

γ_{M2} partial factor for resistance to tensile rupture, including the net section in bolted construction

γ_{M4} partial factor for resistance to cyclic plasticity

γ_{M5} partial factor for resistance of connections

γ_{Mf} partial factor for resistance to fatigue

Membrane stress resultants in a plate (see Figure 3.4)

n_x membrane direct stress resultant that is the force per unit width acting in the x direction in the plane of a plate

n_y membrane direct stress resultant that is the force per unit width acting in the y direction in the plane of a plate

n_{xy} membrane shear stress resultant that is the shear force per unit width acting in the plane of a plate

Membrane stresses in a plate (see Figure 3.4):

σ_{mx} membrane normal stress in the *x*-direction due to a membrane normal stress resultant per unit width n_x

σ_{my} membrane normal stress in the *y*-direction due to membrane normal stress resultant per unit width n_y

τ_{mxy} membrane shear stress due to membrane shear stress resultant per unit width n_{xy}

Bending and twisting stress resultants in a plate (see Figure 3.5)

m_x bending moment per unit width inducing normal stresses in the *x* direction in the plane of a plate

m_y bending moment per unit width inducing normal stresses in the *y* direction in the plane of a plate

m_{xy} twisting bending moment per unit width inducing shear stresses in the plane of a plate

Transverse shear stress resultants in a plate (see Figure 3.5)

q_x transverse shear force per unit width associated with bending stresses in the x direction

q_y transverse shear force per unit width associated with bending stresses in the y direction

Bending and shear stresses in a plate due to bending (see Figure 3.5)

σ_{bx} bending stress in the *x* direction due to bending moment per unit width m_x

σ_{by} bending stress in the *y* direction due to bending moment per unit width m_y

τ_{bxy} shear stress due to the twisting moment per unit width m_{xy}

Transverse shear stresses in a plate

τ_{bxz} shear stress due to transverse shear forces per unit width q_x associated with bending

τ_{byz} shear stress due to transverse shear forces q_y associated with bending

NOTE 3 In general, there are eight stress resultants in a plate at any point. In all plates within the scope of this document, the transverse shear stresses τ_{bxz} and τ_{byz} due to q_x and q_y are negligible compared to the other components of stress, and therefore they can be disregarded in the resistance assessment of an individual plate, though they are required for the analysis of the stress state. For the resistance assessment, only six stress resultants at every point are required.

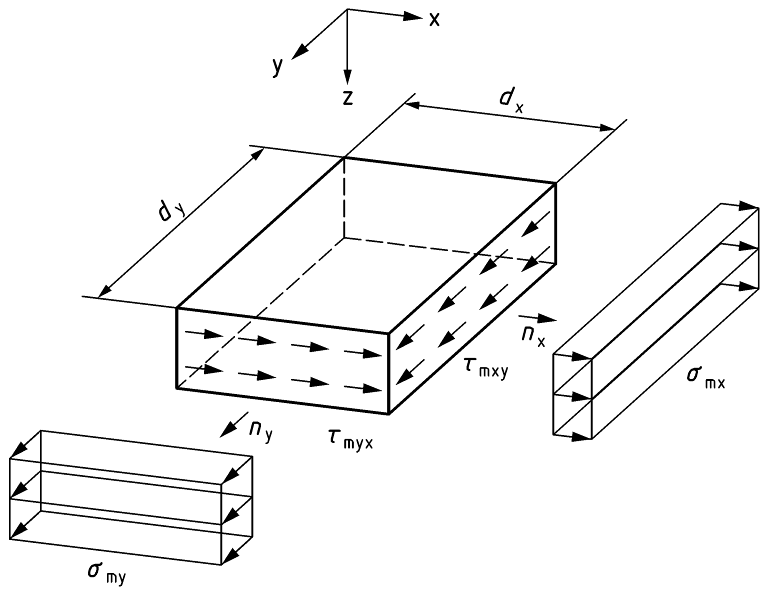


Figure 3.4 — Membrane stresses and membrane stress resultants in a plate

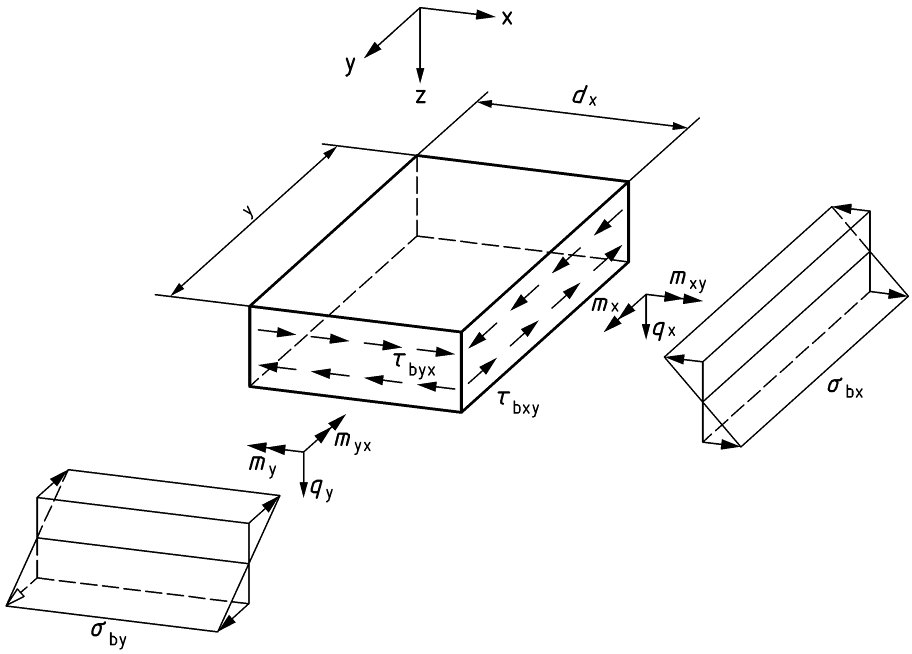


Figure 3.5 — Bending stresses and bending moments in a plate

4 Basis of design

4.1 General

(1) The design of steel structures shall be in accordance with the general rules given in EN 1990 and EN 1991 (all parts) and the specific design provisions for steel structures given in the other relevant parts of EN 1993-1 (all parts).

(2) Steel structures designed according to this document shall be executed according to EN 1090-2 and EN 1090-4 with construction materials and products used as specified in the relevant parts of EN 1993, or in the relevant material and product specifications.

(3) A plate or a plate assembly shall be designed against the ultimate limit states defined in 7.1 and against serviceability limit states in accordance with its intended use and the relevant application or product standards.

(4) This standard is intended for use in conjunction with EN 1993-1-1, EN 1993-1-2, EN 1993-1-3, EN 1993-1-4, EN 1993-1-5, EN 1993-1-9, EN 1993-1-14 and the relevant application parts of EN 1993, which include EN 1993-4-1 for silos.

(5) A plate assembly may be proportioned using design assisted by testing. Where appropriate, the requirements are set out in the appropriate application standard.

NOTE Where design is assisted by testing, additional information and application rules can be given in a National Annex.

(6) All actions should be introduced using their design values according to EN 1990.

(7) Where a stiffened plate assembly is subdivided into individual plate or panels the boundary conditions assumed for stiffeners in the design calculations should be recorded in the drawings and project specification to ensure that the connections have appropriate capacity.

4.2 Reliability management

(1) The execution classes for a plate assembly should be selected in accordance with EN 1993-1-1 or in accordance with the appropriate application or product standards.

(2) The rules for ultimate limit state design in this standard are based on a Reliability Class 2 as defined in EN 1990. If different levels of reliability are required, they should be achieved by an appropriate choice of quality management in design and execution according to EN 1990, EN 1090-2 and EN 1090-4. Where an application standard makes provisions for different Reliability Classes, these provisions may be adopted (e.g. EN 1993-4-1).

4.3 Design values of geometrical data

(1) The thickness t of any plate or part of a plate within a plate assembly should be taken as defined in the relevant application standard. If no application standard is relevant, the nominal thickness of the plate should be used, reduced by an appropriate corrosion or abrasion loss.

(2) The middle surface of each plate or panel should be taken as the reference surface for applied loads, unless stated otherwise in definitions of the load in other standards or application rules.

4.4 Geometrical tolerances and geometrical imperfections

(1) Tolerance values for the deviations of the geometry of each plate or panel surface from the nominal values are defined in EN 1090-2 and the relevant product and application standards.

(2) When the limit state of buckling (LS3, see 7.4) is the limit state to be considered, the geometrical tolerances given in EN 1090-2 should be met. The analysis of the plate assembly is not required to include these tolerances as imperfections, except where GMNIA analysis is used.

4.5 Durability

(1) The provisions of EN 1993-1-1 on durability should be used.

4.6 Verification by the partial factor method

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

(2) The partial factors γ_{Mi} for different limit states should be taken from Table 4.1.

Table 4.1 — Partial factors for resistance

Resistance to failure mode	**Relevant γ**
Resistance to plastic limit state or yielding	γ_{M0}
Resistance to instability / buckling	γ_{M1}
Resistance to rupture	γ_{M2}
Resistance to cyclic plasticity	γ_{M4}
Resistance to fatigue	γ_{Mf}

(3) The numerical values for γ_M defined in Table 4.2 are recommended for plates or plate assemblies that are not covered by the provisions of EN 1993-4-1, or where no application standard exists for the form of construction involved, or the application standard does not define the relevant values.

Table 4.2 (NDP) — Values of partial factors for resistance

γ_{M0} = 1,00	γ_{M1} = 1,10	γ_{M2} = 1,25	γ_{M4} = 1,00	γ_{Mf} see EN 1993-1-9

NOTE The values of each of the partial factors γ_M are given in Table 4.2, unless the National Annex gives different values.

5 Materials and geometry

5.1 Material properties

(1) This document covers the design of plates and plate assemblies fabricated from steel conforming to the product standards listed in EN 1993-1-1 and the relevant application standards.

(2) Where cold-formed sheeting or cold-formed stiffeners are used, the material properties of cold formed sheeting and stiffeners should be obtained from EN 1993-1-3 or the appropriate product standard.

(3) The material properties of stainless steels should be obtained from EN 1993-1-4 or the appropriate product standard.

(4) In a computational analysis using materials with a nonlinear stress-strain relationship, the 0,2% proof stress should be used to represent the yield stress f_y in all relevant formulae. The stress-strain curve should be modelled in accordance with EN 1993-1-14.

(5) Where a material with a nonlinear stress-strain curve is involved and a buckling analysis is carried out under stress design (see 8.4 and 9.4) and the special provisions for stainless steel do not apply, the initial tangent value of Young´s modulus E should be replaced by a reduced value. If no better method is available, the linear elastic stress state should be examined and the peak value of the von Mises equivalent stress should be found from the membrane stress components alone at a distance greater than x_e from any boundary, which may be taken as $10t$. The tangent modulus found using a uniaxial tensile test and corresponding to this stress should then be used to estimate the quasi-elastic critical load or quasi-elastic critical resistance.

NOTE Although the thickest plate treated in this document is b/t = 20 and $10t$ is therefore half the plate width, such thick plates are not susceptible to buckling failure so this exclusion distance for buckling is not too restrictive.

6 Structural analysis

6.1 Types of analysis

6.1.1 General

(1) This standard assumes that a complete plate assembly will often be analysed using a global computational analysis (for example, by means of computer programs based on the finite element method). However, for simple plate assemblies, the treatment using static equilibrium and membrane and simple bending analysis (MSBA) is permitted (see Annex A).

(2) One or more of the following types of analysis should be used, see Table 6.1, depending on the limit state and other considerations:

— Membrane and simple bending analysis (MSBA), see 6.1.2;

— Linear elastic structural analysis (LA), see 6.1.3;

— Linear elastic bifurcation analysis (LBA), see 6.1.4;

— Geometrically nonlinear elastic analysis (GNA), see 6.1.5;

— Materially nonlinear analysis (MNA), see 6.1.6;

— Geometrically and materially nonlinear analysis (GMNA), see 6.1.7;

— Geometrically and materially nonlinear analysis with imperfections included (GMNIA), see 6.1.8.

Table 6.1 — Types of analysis for plate or panels and plate assemblies

Type of analysis	**Treatment**	**Material law**	**Geometry**
Membrane and simple bending analysis (MSBA)	membrane equilibrium for membrane forces; simple bending treatment for forces normal to the plates	static equilibrium alone	perfect
Linear elastic analysis (LA)	linear bending and stretching	linear	perfect
Linear elastic bifurcation analysis (LBA)	linear bending and stretching	linear	perfect
Geometrically nonlinear elastic analysis (GNA)	nonlinear	linear	perfect
Materially nonlinear analysis (MNA)	linear bending and stretching	ideal elastic-plastic	perfect
Geometrically and materially nonlinear analysis (GMNA)	nonlinear	nonlinear	perfect
Geometrically and materially nonlinear analysis including imperfections (GMNIA)	nonlinear	nonlinear	imperfect

6.1.2 Membrane and simple bending analysis (MSBA)

(1) A membrane and simple bending analysis may be used to treat each plate or panel as separate, with a simple one-way beam bending treatment as a horizontal strip in each plate or panel and only simple end shear forces in one plate being transmitted as membrane forces into the adjacent plate or panel.

(2) A membrane and simple bending analysis should only be used provided that the plate junctions are appropriate for transfer of the forces in the plates into support reactions without causing significant local stress effects. For definitions and notes on boundary conditions, see 6.2.4.

(3) A membrane and simple bending analysis does not, in general, fulfil the compatibility of deformations at boundaries or within the plate or panel or between plate or panels either of different shape or that are subjected to different patterns of loading. However, the resulting membrane forces satisfy the equilibrium requirements for the ultimate limit state of plastic failure (LS1).

6.1.3 Linear elastic structural analysis (LA)

(1) Linear elastic structural analysis treats all components as having a linear elastic material law and assumes that the displacements of the plate or plate assembly are governed by small deflection theory (unchanged geometry under load).

(2) An LA analysis satisfies compatibility in the deformations between plate or panels as well as equilibrium. The resulting field of membrane and bending stresses satisfies the requirements for cyclic loading limit states (see 7.3 and 7.5).

(3) This analysis may be undertaken using algebraic formulae (see Annexes A and B) or a computational analysis.

(4) Where a computational analysis is undertaken, the modelling, mesh, validation and verification criteria of EN 1993-1-14 should be met.

6.1.4 Linear elastic bifurcation analysis (LBA)

(1) The result of an LBA analysis provides the elastic critical buckling resistance, which can be interpreted as a load amplification factor R_{cr} on the design value of the loads F_{Ed} and can be used in the verification of limit state LS3. Linear elastic material law and small-deflection theory are used. The lowest buckling eigenvalue $\mathrm{R_{cr}}$ is the lowest load amplification factor on the loads $\mathrm{F_{Ed}}$ at which the considered structure or component may deform into an elastic buckling mode, assuming no change of geometry, no change in the direction of action of the loads, and no material degradation. Imperfections of all kinds are ignored. The term elastic critical is reserved in this document for the outcomes of this analysis (linear elastic with unchanged geometry).

(2) This analysis may be undertaken using a computational analysis or algebraic formulae.

(3) Where a computational analysis is undertaken, the modelling, mesh, validation and verification criteria of EN 1993-1-14 should be met.

6.1.5 Geometrically nonlinear elastic analysis (GNA)

(1) A GNA analysis satisfies both equilibrium and compatibility of the deflections under conditions in which the change in the geometry of the elastic structure caused by loading is included. The resulting field of stresses satisfies the requirements for cyclic loading limit states (LS2 and LS4). It is only computational.

(2) Where changes of geometry caused by the loads produce significant redistributions in the elastic stress state, a GNA analysis may be used for determining the underlying equilibrium state while checking limit state LS1.

(3) Where compression or shear stresses are predominant in some part of the plate assembly, a GNA analysis delivers the nonlinear elastic ultimate or buckling load of the perfect structure, including changes in geometry, that may be of assistance in checking the limit state LS3.

(4) Where this analysis is used for a buckling load evaluation, the eigenvalues of the system should be checked to ensure that the numerical process does not fail to detect a bifurcation in the load path.

(5) The modelling, mesh, validation and verification criteria of EN 1993-1-14 should be met.

6.1.6 Materially nonlinear analysis (MNA)

(1) The result of an MNA analysis gives the plastic reference load, which can be interpreted as a load amplification factor R_{pl} on the design value of the loads F_{Ed}. This analysis provides the plastic reference resistance ratio R_{pl}.

(2) An MNA analysis may be used to verify limit state LS1.

(3) This analysis may be undertaken using a computational analysis or algebraic formulae (see Annex C).

(4) Where a computational analysis is undertaken, the modelling, mesh, validation and verification criteria of EN 1993-1-14 should be met.

(5) An MNA analysis may be used to give the plastic strain increment $\Delta\varepsilon$ during one cycle of cyclic loading that may be used to verify limit state LS2.

6.1.7 Geometrically and materially nonlinear analysis (GMNA)

(1) The result of a GMNA analysis gives the geometrically and materially nonlinear maximum load of the perfect structure and the plastic strain increment that may be used for checking the limit states LS1 and LS2. It is only computational.

(2) Where compression or shear stresses are predominant in some part of the plate assembly, a GMNA analysis gives the elastic-plastic buckling or ultimate load of the perfect structure that may be of assistance in checking the limit state LS3.

(3) Where this analysis is used for a buckling load evaluation, the eigenvalues of the system should be checked to ensure that the numerical process does not fail to detect a bifurcation in the load path.

(4) The modelling, mesh, validation and verification criteria of EN 1993-1-14 should be met.

6.1.8 Geometrically and materially nonlinear analysis with imperfections included (GMNIA)

(1) A GMNIA analysis is used in cases where compression or shear stresses are dominant in the plate assembly. It delivers elastic-plastic buckling or ultimate loads for the imperfect structure, which may be used for checking the limit state LS3.

(2) The imperfections included in the analysis should be compatible with the tolerance requirements of EN 1090-2 and appropriate product standards.

(3) Consideration should also be given to other imperfections, such as residual stresses, wear and corrosion.

(4) Where this analysis is used for a buckling load evaluation, the eigenvalues of the system should be checked to ensure that the numerical process does not fail to detect a bifurcation in the load path.

(5) Where this analysis is used for a buckling load evaluation, an additional GMNA analysis of the perfect plate assembly should always be conducted to ensure that the degree of imperfection sensitivity of the structural system is identified.

(6) The modelling, mesh, validation and verification criteria of EN 1993-1-14 should be met, but the imperfections should be defined according to this document.

NOTE Further guidance on the use of GMNIA analysis is given in EN 1993-1-14 and EN 1993-1-6.

6.2 Modelling of a plate assembly

6.2.1 General

(1) Most plate assemblies subject to transverse loads will behave in a complex manner, depending in particular on the method of support. Unless modelling simplifications can be made with confidence, it is recommended that a global computational analysis is used to determine the stress state throughout the structure.

6.2.2 Plate assembly

(1) Each plate in a plate assembly should be represented by its middle surface.

(2) Imperfections in the plates may be ignored, except when verifying the buckling limit state (LS3).

(3) An assembly of plates or panels should not be subdivided into separate plates for analysis unless the boundary conditions for each plate are chosen in such a way as to represent interactions between them in a manner that leads to safe estimates of resistance.

(4) The overall stability of the complete plate assembly structure should be verified as detailed in the relevant application standards of EN 1993.

(5) Base beams are sometimes used to transfer local support forces into a plate assembly. In the assessment of limit state LS3, these should not be separated from the plate assembly that they support.

NOTE 1 The base beam flexural stiffness is almost always lower than the membrane stiffness of the plate above it in compatible deformation. Only a sufficiently stiff base beam can produce an effective support that can achieve a relatively uniform load transfer to a flat plate stressed in its own plane.

NOTE 2 The torsional flexibility of a base beam can lead to a significant reduction in its effective stiffness in supporting a plate, even when it is located symmetrically beneath the plate that it supports.

NOTE 3 In most applications where discrete supports are used beneath a plate assembly, the force transfer is by in-plane shear forces, which can be high adjacent to the supports and vertical joints.

(6) If a plate or panel that has discrete stiffeners attached to it and is treated by global computational analysis of the plate assembly, each individual stiffened plate may be treated as an orthotropic uniform plate provided that the stiffener separation is small compared with the transverse dimension of the plate.

(7) If a plate or panel that is corrugated (vertically or horizontally) is treated by global computational analysis, each individual corrugated plate may be treated as an orthotropic uniform plate provided that the corrugation wavelength is small compared with the transverse dimension of the plate.

NOTE No provisions are made within this document for the detailed analysis of corrugated plates. Useful advice is available in EN 1993-1-3 and EN 1993-4-1.

6.2.3 Treatment of individual plates or panels

(1) Where simplified analyses are used for the plate assembly, the boundary conditions assumed for each plate should lead to safe assessments of the resistance of individual plates and their connections (i.e. restrained or unrestrained individual plates). If this condition is met, a plate assembly may be subdivided into individual plates or panels that may be considered independently. The interactions between the plates or panels should be examined to determine the actions that each plate transfers to its neighbours (see Figures 6.3 and 6.4).

NOTE The above provision refers to simple calculation treatments, not to the results of global computational models of complete assemblies in which these interactions are already included.

(2) To achieve a safe design, it may be necessary to define the boundary conditions of each individual analysed plate in a manner that minimises its assessed resistance, even if this does not satisfy bending equilibrium between plate or panels.

NOTE When considering appropriately safe boundary conditions, care is required with plates under non-uniform loading. While for uniformly loaded simply supported plates the mid-span location probably remains the position of the maximum moment, under non-uniform loading (e.g. hydrostatic or geostatic loads) moments at fixed edges can be significant.

(3) Where an in-plane tensile membrane stress resultant exceeds n_{lim}, any eccentricity or step in the middle surface of a plate should be considered in a fatigue (LS4) or cyclic plasticity assessment (LS2), due to the significant additional bending arising from the eccentric load path.

(4) The value of n_{lim} should be assessed as

$$n_{\text{lim}} = 0{,}2\frac{m_p}{e} = 0{,}05\left(\frac{t}{e}\right)tf_y \tag{6.1}$$

where

m_p is the full plastic moment per unit width of plate;

t is the thickness of the thinner plate at the step;

e is the eccentricity between the middle surfaces of the two joined plates.

NOTE The value of n_{lim} chosen in Formula (6.1) ensures that the local bending moment is less than 0,2 m_p, which indicates that there is effectively no reduction in the static bending resistance of the plate and the increase in elastic surface stress is limited to 0,3f_y.

(5) A corrugated plate may be treated as an orthotropic uniform plate, acting in one way bending (see EN 1993-4-1).

(6) An isolated hole in a plate may be neglected in computational modelling, provided its largest dimension is smaller than the lesser of $5t$ and either $b/10$ or $b_1/10$ and the hole is not within the exclusion distance x_e of an edge as defined by 8.2.7.1 (2).

6.2.4 Boundary conditions

(1) The boundary conditions assumed for an individual plate should be chosen to ensure that they achieve an appropriate model of the real construction. Special attention should be given not only to the constraint of displacements normal to each plate (deflections), but also to the constraint of the displacements in the plane of the plate because of the significant effect these can have on both strength and buckling resistance.

NOTE Further information on boundary conditions in computational models is given in EN 1993-1-14.

(2) Support boundary conditions should be checked to ensure that they do not cause excessive non-uniformity of transmitted forces or introduced forces that are eccentric to a plate middle surface. For the detailed application of this rules for silos, further information may be found in EN 1993-4-1.

(3) Rotational restraints at plate or panel boundaries should be included in modelling for limit states LS2 and LS4 (see Clause 7), but may be neglected in modelling for limit states LS1 and LS3.

(4) In computational analyses and in selecting formulae from Annexes A to C, the appropriate boundary conditions should be used in analyses for the assessment of limit states according to the conditions shown in Table 6.4. For the special conditions needed for buckling calculations, see 7.4, 8.4 or 9.4 as appropriate.

Table 6.4 — Boundary conditions for plate edges in the *x*-*y* plane

Boundary condition code	**Simple term**	**Description**	**Displacements**		**Rotation**
			out-of-plane	in-plane	about the edge
BC1r	Clamped	out-of-plane displacements restrained in-plane displacements restrained rotation restrained	$w = 0$	$u = 0$ $v = 0$	$\phi = 0$
BC1f		out-of-plane displacements restrained in-plane displacements restrained rotation free	$w = 0$	$u = 0$ $v = 0$	$\phi \neq 0$
BC2r		out-of-plane displacements restrained in-plane displacements free rotation restrained	$w = 0$	$u \neq 0$ $v \neq 0$	$\phi = 0$
BC2f	Pinned	out-of-plane displacements restrained in-plane displacements free rotation free	$w = 0$	$u \neq 0$ $v \neq 0$	$\phi \neq 0$
BC3	Free edge	out-of-plane displacements free in-plane displacements free rotation free	$w \neq 0$	$u \neq 0$ $v \neq 0$	$\phi \neq 0$
NOTE It is assumed that restraint of an edge displacement in the plane of the plate restrains both displacements normal to the edge and parallel to the edge.					

6.2.5 Modelling of plate junctions

(1) When a global computational analysis is used, care should be taken in modelling of the boundaries between adjoining plates (termed junctions) to ensure that the transfer of forces and moments is appropriately represented, paying attention to the structural detailing of the joint (see EN 1993-1-8).

NOTE A wide range of structural details is used to join adjacent plates. These connections can be flexible or stiff according to the specific details of the geometry. The stiffness of the connection can lead to significantly different results in the transfer of forces and moments between the plate or panels.

(2) The ultimate limit state of cyclic plasticity should be carefully considered when detailing the joints between plate or panels.

(3) Unless special provision is made, the junctions should be designed to transmit the full forces and moments associated with rigid joints. Flexible joints should be designed with an appropriate rotation capacity and adequate resistance.

NOTE Unless the forces and moments associated with fully rigid joints are adopted in the design, there is a possibility of failure of the joint itself by plastic failure (LS1) or by cyclic plasticity (LS2).

(4) In assemblies of stiffened plates, where full continuity of the stiffeners across joints is assumed in models, this should be ensured by appropriate detailing (see EN 1993-1-8).

6.3 Modelling of actions and environmental influences

(1) Loads and other actions should be assumed to act at the plate middle surface. Eccentricities of load should be represented by static equivalent forces and moments at a plate middle surface.

(2) Unless an appropriate justification can be given, local actions should not be represented by equivalent uniform loads.

(3) The modelling should account for all potential conditions that may affect the structure, of which the following should be considered where relevant:

— local settlement under plate edges;

— local settlement under discrete supports;

— uniformity / non-uniformity of support of the structure;

— thermal differentials from one side of the structure to the other;

— thermal differentials from the inside to the outside of the structure;

— connections to other structures;

— conditions during erection.

6.4 Simplified analysis methods for plate assemblies under general loads

6.4.1 General

(1) The internal forces or stresses of a plate or plate assembly loaded by both out-of-plane and in-plane loads may be determined using the simplified models defined here.

(2) The generic form of a plate assembly considered by this standard is illustrated in Figure 6.1, though not all illustrated parts will be present in each structure. This assembly consists of plates with rectangular, trapezoidal or triangular shapes, where each plate or panel has at least one axis of symmetry.

(3) The interactions between plate or panels that have shapes other than rectangular have more complex forms, and may usefully be treated using global computational analysis.

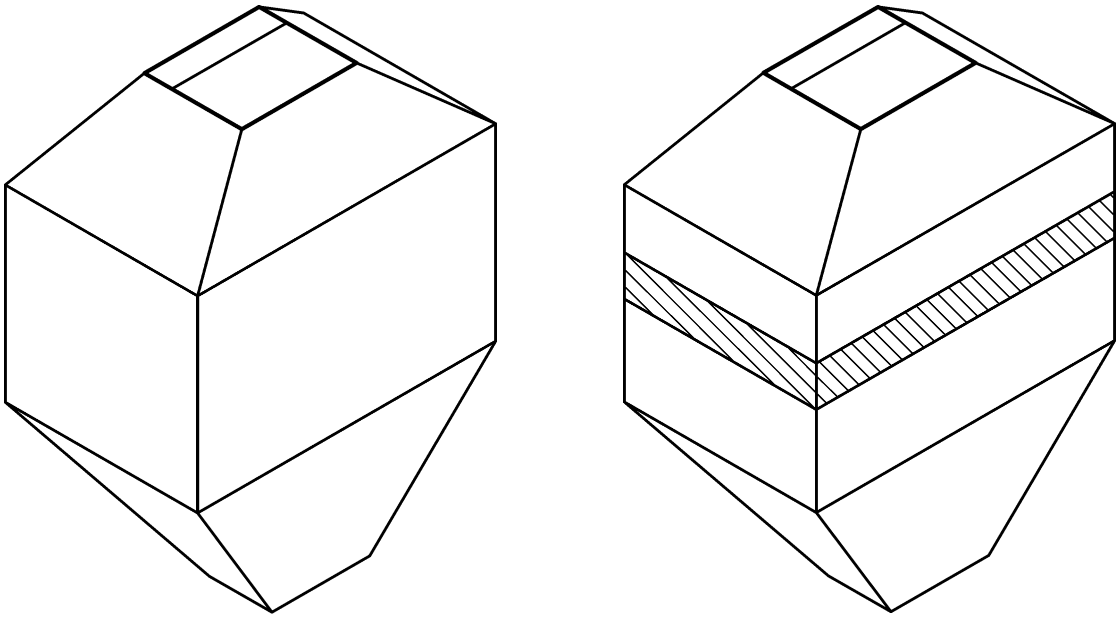
(4) The transverse loads on the plates of a rectangular assembly may be treated as supported on the plate edges in such a way that each horizontal strip of plate subject to relatively uniform pressure may be deemed to act so that the end shear on the strip is transmitted to the adjacent plate as a membrane force (see Figure 6.2).

(5) The interactions in a transversal slice of a plate assembly between plate or panels in bending may be approximately treated using a simple frame analysis. Such an analysis assumes that each plate or panel has a length and mean stiffness that permits the interactions between the frame members, and the joints between them, to be accurately represented by this simple frame treatment (see Figure 6.3).

(6) The end shear on the strip is transmitted to the adjacent plate as a membrane force (see Figures 6.3 and 6.4).

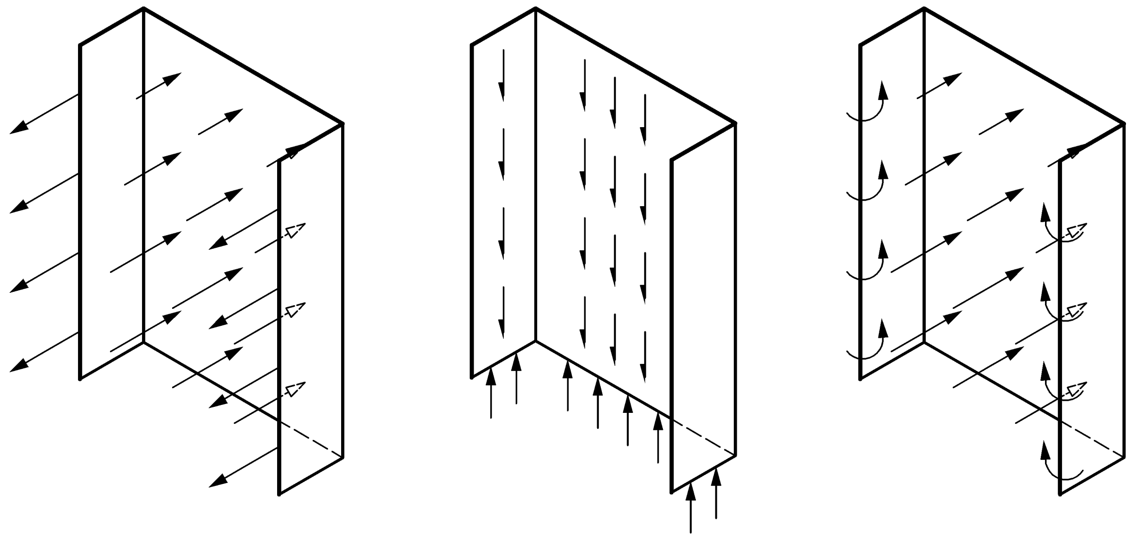
(7) The interactions that occur between triangular and trapezoidal plate or panels, both with each other and with other parts of the plate assembly, require special treatment (see 6.5.2).

(8) In the vertical plane, the transverse loads and frictional shears on the plate or panels of a rectangular assembly may be treated as supported on the plate edges in such a way that each vertical or inclined strip of plate may be deemed to act so that the end membrane force and end shear on the strip are transmitted to the adjacent plate as a corresponding membrane force and shear (see Figure 6.3 and 6.4).



a) Assembly of unstiffened plates b) Transversal slice extracted

Figure 6.1 — Unstiffened plate assembly and horizontal slice



a) Membrane forces from pressures normal to the plate

b) Membrane forces from surface tractions

c) Bending moments within plate elements

Figure 6.2 — Membrane forces and bending moments arising from normal pressures and frictional tractions on plate or panels

(9) Plate assemblies are commonly supported at discrete points, frequently beneath the vertical boundaries between plate or panels (see Figure 6.5). Where a single plate forms the side of an assembly and the assembly is discretely supported at the two ends of this plate (see 6.3.2 and Figure 6.5), this plate should be analysed as a deep beam spanning between supports if its aspect ratio ψ is greater than 0,2.

(10) A structural beam member at the base of the plate is generally ineffective as a means of supporting the plate, so this structural concept should not be used. If a structural beam is used (Figure 6.5), its bending stiffness relative to the membrane stiffness of the plate above should be carefully evaluated to determine whether it can act as intended.

NOTE The in-plane membrane stiffness of even a thin plate is generally very high compared to the bending stiffness of any supporting structural element. An analysis that considers the true stress pattern developing in the plate is required to ensure that the forces in the plate are transmitted as expected and that buckling of the plate does not arise as a consequence of the actual developed stress pattern.

(11) Where a plate assembly is supported at its corners (Figure 6.5), the deep beam action means that each side plate acts as a web. The upper edge of each side plate should be attached to a stiff structural member to provide an effective flange of the deep beam (Figure 6.5 b) and c)). The stability of the upper edge member should be carefully checked for buckling out of the plane of the plate (see EN 1993-1-1).

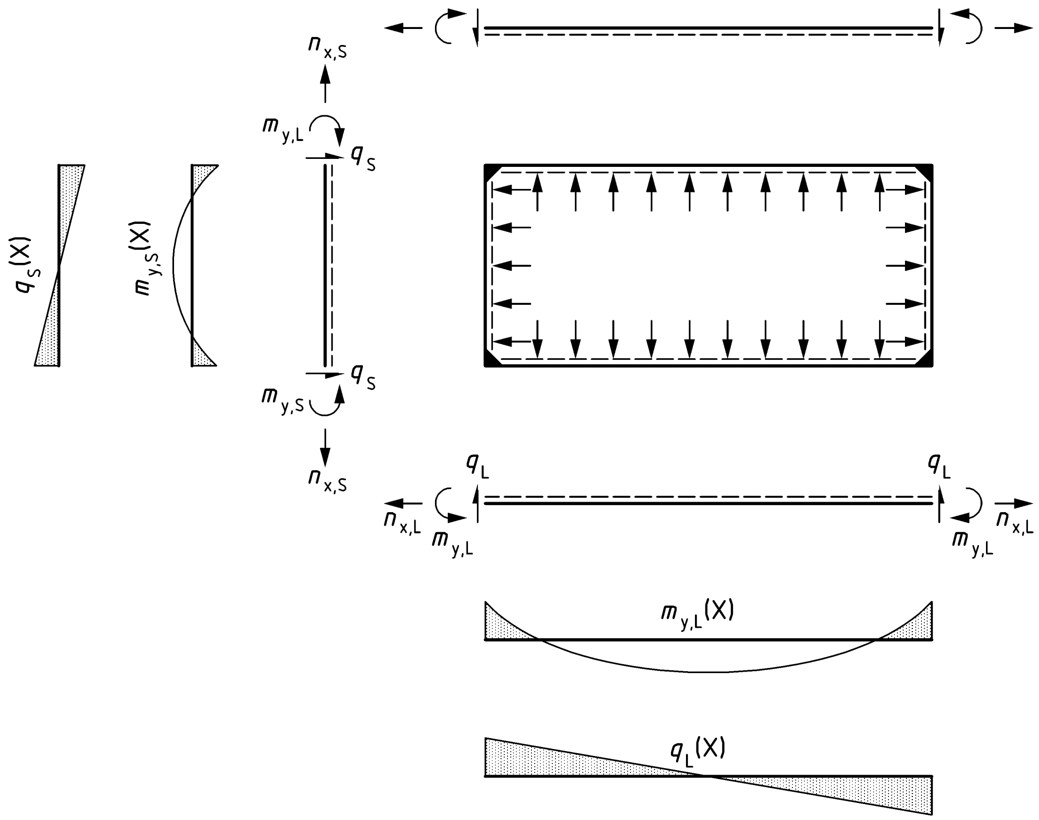
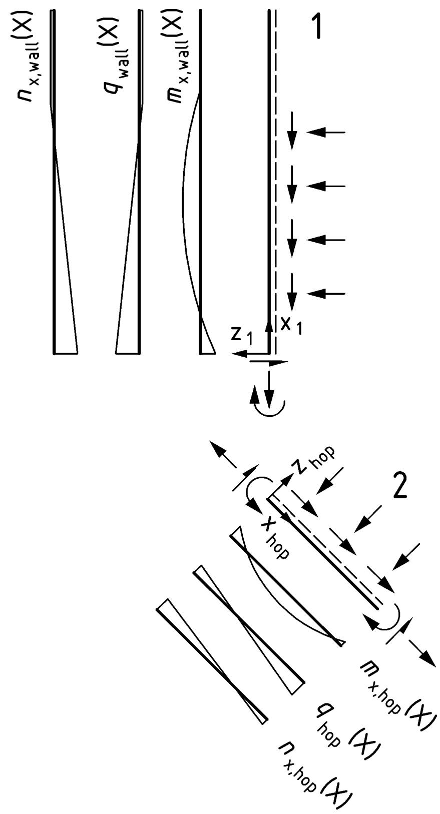


Figure 6.3 — Membrane force, shear and bending transmission in a transversal (horizontal) slice through a vertically standing plate assembly

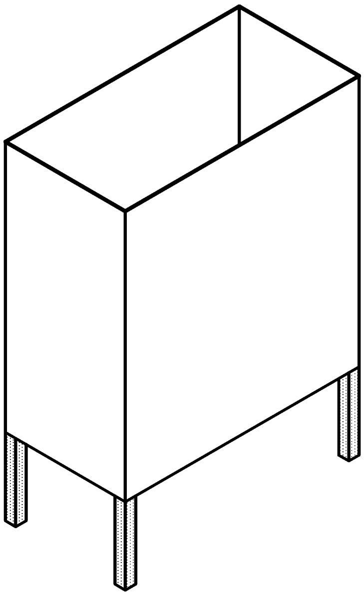


Key

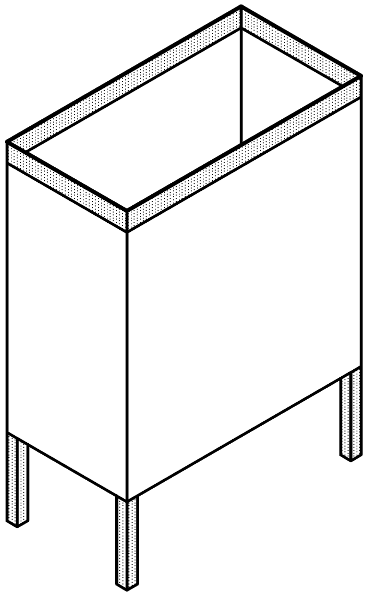
1 wall panel

2 hopper

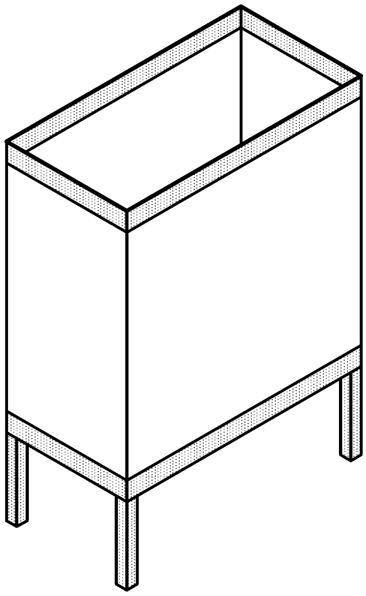
Figure 6.4 — Membrane force, shear and bending transmission between wall panels and hopper in a vertically standing, rectangular plate assembly of the type in Figure 6.1



a) Simple supports under box



b) With eaves beam



c) Beam support under walls with eaves beam

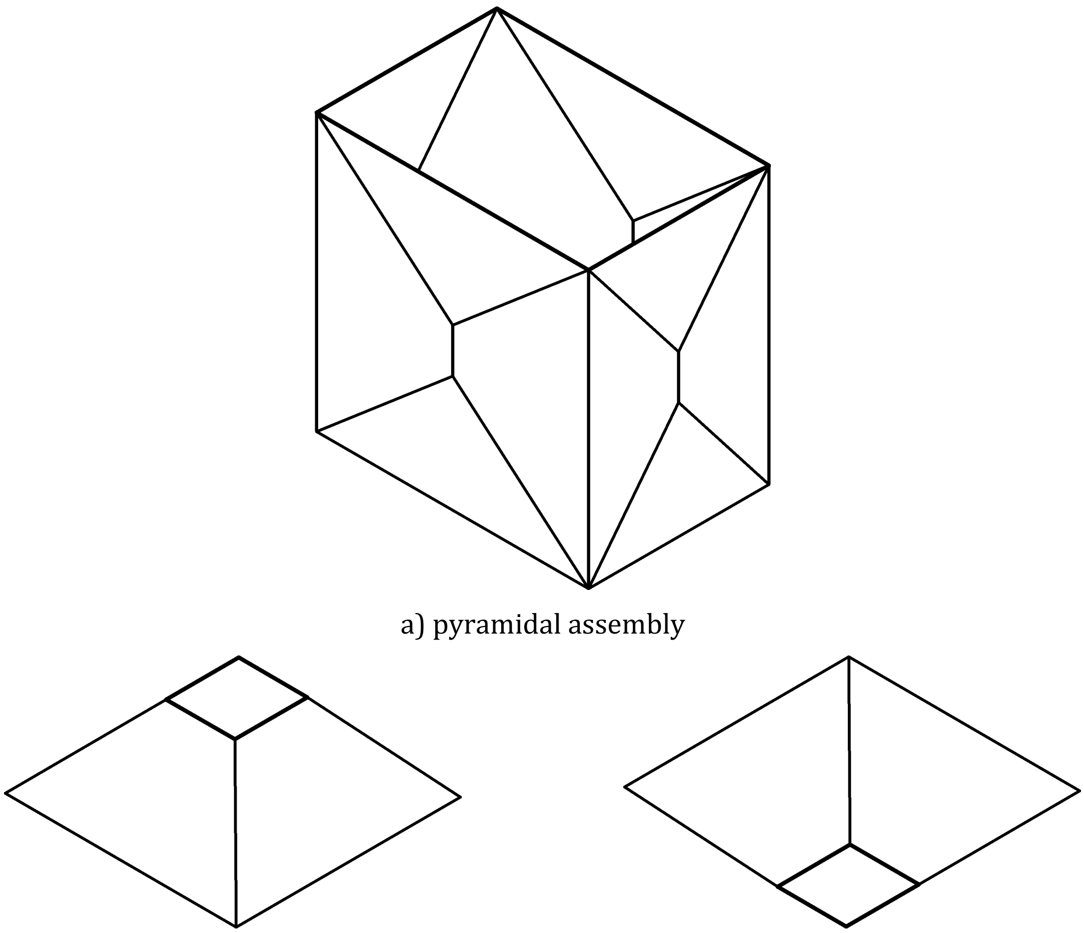
Figure 6.5 — Schematic illustrations of support conditions for rectangular assemblies

6.4.2 Pyramidal assemblies

(1) The transverse load on a trapezoidal or triangular plate in a pyramidal assembly may be treated as a uniform pressure and traction loads throughout the plate (see Annexes A and B). The value should be chosen as the highest pressure predicted to act on any part of the plate's surface according to EN 1991-4.

(2) The analysis of forces and bending moments in trapezoidal or triangular plates in a pyramidal assembly may be treated using the simple analyses of Annex A, may alternatively be found using simple finite element analyses of individual plates, or a full finite element analysis of the complete assembly.

NOTE This arrangement of plates produces a very stiff and strong structure, leading to significantly reduced forces and moments in the plate or panels, that require evaluation using the provisions presented here.



a) pyramidal assembly

b) pyramidal roof

c) pyramidal hopper

Figure 6.6 — Pyramidal assemblies

6.5 Analysis of individual plates or panels

6.5.1 General

(1) The stress resultants in an individual plate or panel of a plate assembly should be evaluated when at least a preliminary evaluation of the actions on it from other plates has been made.

(2) The required treatment of an individual plate or panel depends significantly on whether the element is unstiffened or stiffened. The following paragraphs define appropriate treatments for each element type.

6.5.2 Analysis of unstiffened plates or panels

(1) Unstiffened plates or panels may be analysed by computational analysis, simplified equivalent beam models, or elastic plate analysis using the formulae for maximum bending moments under the commonest loading conditions in Annex B.

(2) An unstiffened rectangular plate under out-of-plane loads may be modelled as an equivalent beam in the direction of the dominant load transfer, provided that the following conditions are fulfilled:

— for rectangular plates, the out-of-plane distributed loads should be either constant or linearly varying in the long dimension a;

— for triangular and trapezoidal plates, the out-of-plane distributed loads should be either constant or linearly varying along the axis of symmetry;

— the strength, stability and stiffness of the frame, beam or adjacent plate or panel which provides the boundary supports for the plate or panel should be shown to meet the needs of the plate or panel treated as an equivalent beam.

NOTE This treatment is very conservative if the plate aspect ratio $\psi = b/a$ is close to 1,0.

(3) The internal forces and moments of the equivalent beam should be determined using an elastic or plastic analysis as defined in EN 1993-1-1.

(4) If an in-plane compressive membrane stress resultant acts in the direction of the span of the equivalent beam throughout the length of the plate, the first order deflection due to the out-of-plane loads is approximately congruent to the plate buckling mode. In this case, the amplification of the first order moments by the in-plane compression should be taken into account as a buckling limit state condition (LS3), see 7.4 and 8.4.2.

6.5.3 Analysis of uni-directionally stiffened plate or panels

(1) Uni-directionally stiffened plate or panels may be analysed by either computational analysis or simplified beam models. Both the transverse forces acting directly on the stiffened plate and the global effects acting at the interfaces with other stiffened plates should be considered in the simplified analysis, see Figure 6.7.

NOTE Figure 6.7 shows a typical configuration for a hydraulic lock gate. Significant compression in longitudinal direction is present in each of the two stiffened plates shown due to the global structural arrangement, boundary conditions and loading.

(2) A plate or panel that is stiffened in only one direction may be modelled as a series of adjacent beams, provided that the boundary conditions of the ends of each stiffened zone provide the required support. The resulting beam model should take account of any axial stresses from the plate.

(3) The uni-directional stiffener acts integrally with the plate and acts as an equivalent T section or I section beam (depending on the stiffener section). The effective width b_{eff} of the plate that acts with the stiffener may be taken as the lesser of:

— the spacing of the parallel stiffeners;

— $15t\sqrt{(235/f_y)}$ with f_y expressed in MPa;

— $L/50$;

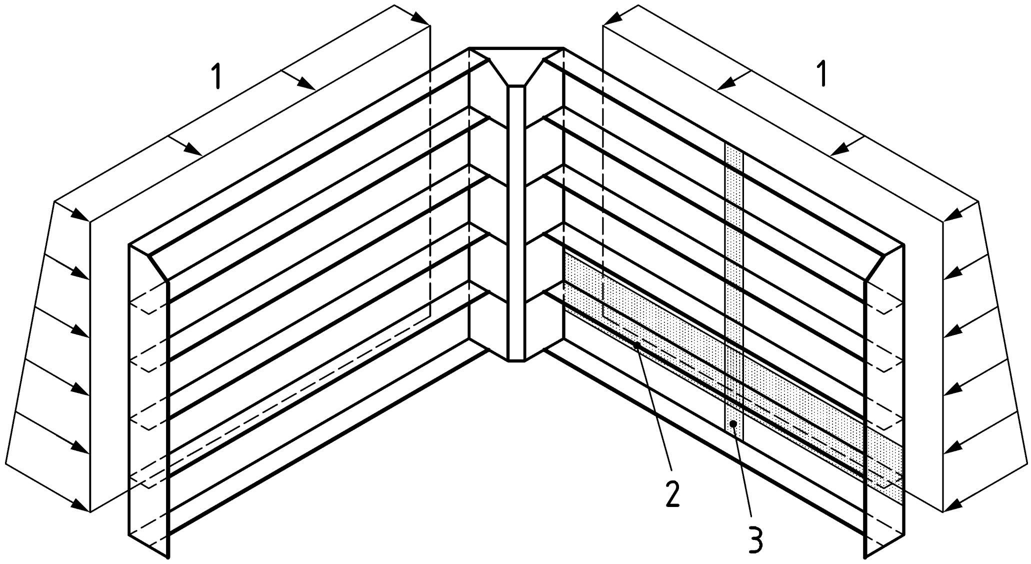
where L is the span of the stiffened plate between supports.

(4) A more efficient design may be achieved using the provisions for the effective width allowing for shear lag according to EN 1993-1-5.

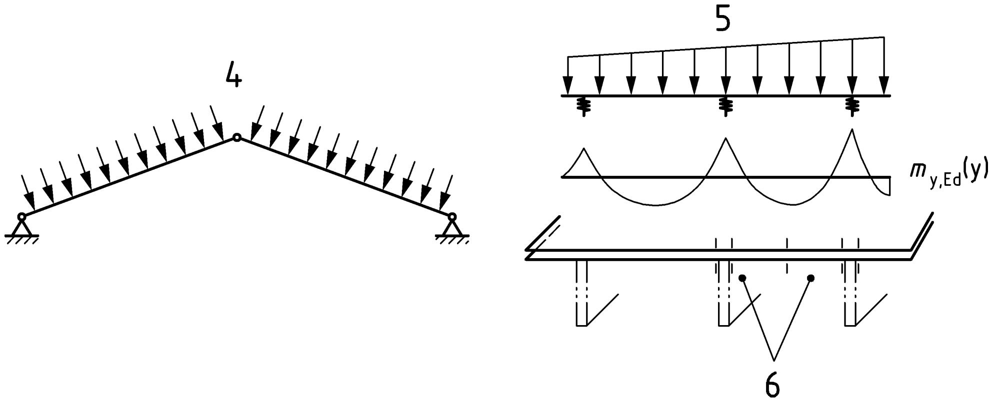
(5) The stresses in the transverse direction may also be determined using a beam model on rigid or elastic supports.

(6) The combined effect of the stresses determined for the longitudinal and transverse beam models should be included in the ultimate limit state in accordance with Clause 9.

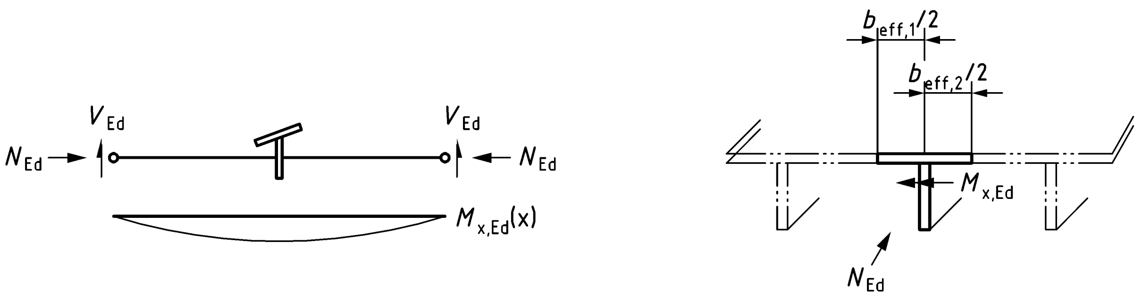
NOTE In stress-based design, the combined effects are usually verified at individual critical sections, see Figure 6.7.



a) global view



b) simplified two-dimensional model



c) simplified model for a single longitudinal stiffener

Key

1 transverse load p_z

2 longitudinal stiffener with effective plate

3 transverse plate strip (unit width)

4 longitudinal direction

5 transverse direction

6 critical sections

Figure 6.7 — Simplified analysis of uni-directionally stiffened plates

(7) Where the stiffener is placed on the side opposite to the transverse load on the plate and no axial force is introduced directly into the stiffener from external sources, no buckling check of the stiffener is required.

6.5.4 Analysis of bi-directionally stiffened plates or panels

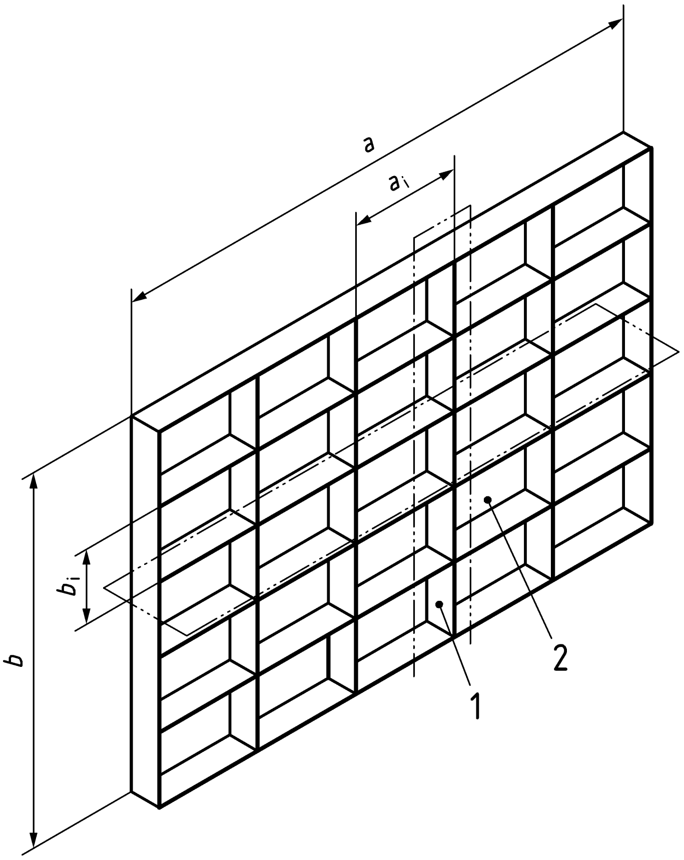
(1) A plate or panel that is stiffened in both directions may be modelled either by computational analysis or as a grillage if it has a rectangular shape and is stiffened in both the transverse and longitudinal directions, see Figure 6.8.

NOTE 1 The chosen value for the torsional stiffness of the grillage members can significantly affect the results of this analysis.

NOTE 2 Where the bi-directionally stiffened plate does not meet the geometric requirements for analysis as a grillage, no simple treatment is available in this document.

(2) The stiffeners of a bi-directionally stiffened plate or panel that is modelled and analysed as a grillage may be verified as individual cross-sections using the rules of EN 1993-1-1, with the effective widths taken from 6.5.3(3) or EN 1993-1-5.

(3) The stresses in the stiffened plate (sub-panels) should be calculated as the superposition of membrane stresses due to the beam action of the longitudinal or transverse stiffener within the effective width, and the bending and membrane stresses due to bending in the sub-panel between the stiffeners.



Key

1 transverse stiffener

2 sup-panel

Figure 6.8 — Typical rectangular bi-directionally stiffened plate assembly

6.6 Analysis by computational modelling

(1) The analysis of a plate assembly by computational modelling should be conducted using the rules of EN 1993-1-14.

NOTE A linear elastic analysis (LA) is usually adequate for the analysis of unstiffened, uni-directionally stiffened and bi-directionally stiffened plate assemblies, but can produce a very conservative treatment.

7 Ultimate limit states for plate assemblies

7.1 General

(1) Plates and plate assemblies shall be designed against the four ultimate limit states described in the following sub-clauses.

7.2 Plastic failure limit state (LS1)

(1) The plastic failure ultimate limit state is the condition in which a part of the structure either develops excessive plastic deformations, as in a plastic mechanism, or suffers rupture.

NOTE At this limit state, the loads or actions (resistance) cannot be increased without exploiting either a significant change in the geometry or strain-hardening of the material.

(2) The limit state of tensile rupture is the condition in which a plate experiences through-thickness tensile failure, leading to separation of the two parts of the plate or separation of two plate or panels from each other at a junction.

NOTE For rupture caused by net section failure at a junction, see EN 1993-1-8.

(3) In verifying the plastic failure state, plastic or partially plastic behaviour of the structure may be assumed (i.e. elastic compatibility considerations may be neglected).

(4) The plastic reference load should be derived from a mechanism based on small deflection theory.

(5) For LS1, the design values of the actions shall be based on the most adverse relevant load combination.

(6) Only those actions that represent loads affecting the equilibrium of the structure need be included.

(7) One or more of the methods of analysis described in 6.1 should be used for the calculation of the design stresses and stress resultants when checking LS1.

(8) In the absence of fastener holes, verification at the limit state of tensile rupture may be assumed to be covered by the check for the plastic failure state. However, where holes for fasteners occur, a supplementary check in accordance with EN 1993-1-1 should be performed.

7.3 Cyclic plasticity limit state (LS2)

(1) The ultimate limit state of cyclic plasticity occurs when repeated alternating cycles of plastic strain cause exhaustion of the plastic capacity of the material at a lower level than under monotonic load. It is a low cycle fatigue restriction.

NOTE The stresses that are associated with this limit state develop under a combination of all actions and the compatibility conditions for the structure.

(2) All variable actions (such as imposed loads and temperature variations) that can lead to yielding, and which might be applied with more than three cycles in the life of the structure, should be accounted for when checking LS2.

(3) In the verification of this limit state, compatibility of the deformations under elastic or elastic-plastic conditions should be considered.

(4) One or more of the following methods of analysis (see 6.1) should be used for the calculation of the design stresses and stress resultants when checking LS2:

— elastic global computational analysis (LA or GNA) to determine the elastic stress range;

— nonlinear global computational analysis (MNA or GMNA) to determine the plastic strain range.

7.4 Buckling limit state (LS3)

(1) The ultimate limit state of buckling is the condition in which all or part of the structure suddenly develops large displacements normal to a plate surface, caused by loss of stability under compressive membrane and/or shear stresses in one or more of the plates or panels.

NOTE In most cases, plate buckling leads to stiffening post-buckling conditions, permitting higher loads to be supported. The initiation of buckling is consequently not as critical as for columns and shells.

(2) The reference linear elastic buckling resistance is derived from a linear bifurcation analysis (LBA) of the plate assembly or plate or panel.

(3) For local plate buckling under membrane stresses, see EN 1993-1-5 in combination with the rules in 8.4 and 9.4.

(4) For flexural, lateral torsional and distortional stability of stiffeners, see EN 1993-1-5.

(5) Either of the following methods of analysis (see 6.1) may be used when checking LS3:

— linear elastic bifurcation analysis (LBA) may be used for plate assemblies under general loading conditions to obtain the elastic critical buckling resistance to be used in the buckling verification of EN 1993-1-5;

— GMNIA using appropriate imperfections and calculated calibration factors, with interpretation of the results supported by the outcomes of MNA, LBA and GMNA calculations.

(6) All relevant load combinations at the design values that induce compressive membrane or shear membrane stresses in the plates should be accounted for when checking LS3.

7.5 Fatigue limit state (LS4)

(1) The limit state of high cycle fatigue should be taken as the condition in which repeated cycles of increasing and decreasing stress caused by variable actions lead to the development and propagation of a fatigue crack.

(2) A fatigue verification according to EN 1993-1-9 should be carried out for structures exposed to high cycle variable actions.

(3) Design values of actions and load spectra that produce stress ranges $\Delta\sigma$ relevant to the fatigue limit state may be specified in EN 1991, in application parts of EN 1993 or in relevant product specifications.

(4) If equivalent constant stress ranges $\Delta\sigma_{e,2,Ed}$ as defined in prEN 1993-1-9:2023, 7.3.2 or 7.3.3 are specified in the documents identified in (2), it should be verified that their definition matches the chosen stress design approach, see (4). In other cases, the design value of equivalent stress range $\Delta\sigma_{e,2,Ed}$ may be calculated according to prEN 1993-1-9:2023, 7.3.4 from the linearly accumulated damage *D* at the notch. The cumulative linear damage model of prEN 1993-1-9:2023, Annex A should be used to calculate *D*.

(4) The stress ranges $\Delta\sigma$ and the stress range spectra resulting from the actions and load spectra specified in (2) should be calculated relevant constructional details or notches in the plate or plate assembly, considering the appropriate design stress methods of prEN 1993-1-9:2023, 6.1(1).

NOTE Constructional details relevant to LS4 are generally found at welded or bolted joints, connections, stiffeners or attachments. A classification of constructional details and the corresponding fatigue resistance values to be used for the chosen design stress method are given in prEN 1993-1-9:2023, Clause 10, Annex B and Annex C for the nominal or modified nominal stress method, hot spot stress method and effective notch stress method, respectively.

(5) The nominal stress method of EN 1993-1-9 may be used only in cases where membrane stresses in the plate middle surface represent the design stress range spectrum in the proximity of the considered notch with sufficient accuracy. In all other cases, the hot spot stress method of prEN 1993-1-9:2023, Annex B or the effective notch stress approach of prEN 1993-1-9:2023, Annex C should be used in fatigue design of transversally loaded plates and plate assemblies.

(6) In determining the elastic stress ranges $\Delta\sigma$, linear elastic analysis (LA) may be used if geometric nonlinearity can be neglected. However, if geometric nonlinearities modify the linear stress distribution significantly, nonlinear elastic analysis (GNA) should be used.

(7) The principal stress with the largest absolute stress range should be used in design, unless its orientation deviates by more than 45° from that of the normal to the notch (weld line or bolt hole edge). In the latter case, the stress components perpendicular to the notch should be used, see prEN 1993-1-9:2023, Annex B.

(8) The partial factors for fatigue design should be taken from EN 1993-1-9.

(9) Multiaxial fatigue loading should be verified in accordance with prEN 1993-1-9:2023, 9.4.

8 Ultimate limit state design of unstiffened plates

8.1 General

(1) Unstiffened plates shall be designed against the limit states LS1 to LS4 as described in this sub-clause, using either stress-based design, design using standard formulae or computational design.

NOTE Sub-panels between stiffeners of stiffened plates are dealt with in Clause 10.

(2) If an unstiffened plate is designed to act in only one-way bending as an equivalent beam, its cross-section resistance should be checked for the combination of in-plane loading and out-of-plane loading using the provisions of EN 1993-1-1.

8.2 Plastic failure limit state (LS1)

8.2.1 General

(1) Only those actions that represent loads affecting the equilibrium of the structure need be included.

(2) The plastic failure limit state may be assessed using any of the following methods:

— Simple design for one-way bending (equivalent beam)

— Stress-based design

— Design by standard formulae

— Design using computational analysis

8.2.2 Design values of resistance

(1) Yield line analysis may be used in the ultimate limit state when membrane tension, compression or shear are less than 10% of the membrane yield resistance. The bending resistance in a yield line should be taken as

$$m_{Rd} = \frac{f_y}{\gamma_{M0}} \frac{t^2}{4} \tag{8.1}$$

(2) The membrane tension or compression resistance should be taken as

$$n_{x,Rd} = n_{y,Rd} = \frac{f_u t}{\gamma_{M2}} \tag{8.2}$$

(3) The membrane shear resistance should be taken as

$$n_{xy,Rd} = \frac{f_u t}{\gamma_{M2}\sqrt{3}} \tag{8.3}$$

(4) Where membrane tension exceeds 10% of the tensile yield resistance, or

$$n_{eq,Ed} > 0{,}1\ n_{x,Rd} \tag{8.4}$$

the bending resistance in a yield line should be reduced to

$$m_{Rd} = \frac{f_y}{\gamma_{M0}} \frac{t^2}{4} \left\{1 - g_{eq}^2\right\} \tag{8.5}$$

in which

$$g_{eq} = \gamma_{M2} \frac{n_{eq,Ed}}{n_{x,Rd}} \tag{8.6}$$

$$n_{eq,Ed} = \sqrt{n_{x,Ed}^2 - n_{x,Ed} n_{y,Ed} + n_{y,Ed}^2 + 3n_{xy,Ed}^2} \tag{8.7}$$

(5) The bending stress resistance corresponding to the yield line bending moment resistance should be obtained from

$$\sigma_{eq,Rd} = \frac{4m_{Rd}}{t^2} \tag{8.8}$$

(6) The effect of fastener holes should be taken into account in accordance with EN 1993-1-1:2022, 8.2.3 for tension and EN 1993-1-1:2022, 8.2.4 for compression.

(7) The design of long bolted joints should account for the non-uniform distribution of forces and stresses to be transferred by the bolts, as well as through the net section of the adjacent plate, following the pertinent rules in EN 1993-1-1 and EN 1993-1-8.

8.2.3 Stress-based design

(1) Design values of relevant membrane and bending stress resultants may be found in the formulae in Annexes A and B.

(2) Where one way bending and membrane forces are used, see 8.2.4.

(3) Where formulae for bending moments in elastic plates are used (see Annex B) with membrane forces from Annex A, see 8.2.5.

(4) Where there is a two-dimensional stress field resulting from a linear elastic analysis, the von Mises equivalent stress $\sigma_{eq,Ed}$ may be determined, as

$$\sigma_{eq,Ed} = \sqrt{\sigma^2_{x,Ed} - \sigma_{x,Ed}\,\sigma_{y,Ed} + \sigma^2_{y,Ed} + 3\tau^2_{xy,Ed}} \tag{8.9}$$

in which

$$\sigma_{x,Ed} = \frac{n_{x,Ed}}{t} \pm \frac{4m_{x,Ed}}{t^2} \tag{8.10}$$

$$\sigma_{y,Ed} = \frac{n_{y,Ed}}{t} \pm \frac{4m_{y,Ed}}{t^2} \tag{8.11}$$

$$\tau_{xy,Ed} = \frac{n_{xy,Ed}}{t} \pm \frac{4m_{xy,Ed}}{t^2} \tag{8.12}$$

with $n_{x,Ed}$, $n_{y,Ed}$, $n_{xy,Ed}$, $m_{x,Ed}$, $m_{y,Ed}$ and $m_{xy,Ed}$ defined in 3.2 (5) and (7).

NOTE The above formulae give a simplified and generally conservative equivalent stress for design.

(5) The exclusion distance x_e defined in 8.2.7.1 may be used to reduce the number of locations at which the test of Formula (8.9) is required to be applied.

NOTE 1 If the calculated von Mises equivalent surface stresses at every point in the plate are used (e.g. as output from a computer program), local stress conditions often dominate and the design will be more conservative than that obtained an evaluation using Formulae (8.9) to (8.12).

NOTE 2 Formulae (8.9) to (8.12) correspond to the Ilyushin yield criterion.

(6) It should be verified that

$$\sigma_{eq,Ed} \le \frac{f_y}{\gamma_{M0}} \tag{8.13}$$

8.2.4 Simple design for one-way bending

(1) If an unstiffened plate is designed as an equivalent beam, the simpler treatment of its cross-section resistance may be checked using the following.

(2) The bending resistance should be taken from Formula (8.1). The membrane tension or compression yield resistance should be taken from Formula (8.2). The reduced bending resistance in the presence of a membrane force should be taken from Formula (8.5) using

$$g_{eq} = \gamma_{M2}\frac{n_{x,Ed}}{n_{x,Rd}} \tag{8.14}$$

(3) The equivalent surface stress should be found as

$$\sigma_{eq,Ed} = \frac{n_{x,Ed}}{t} \pm \frac{4m_{x,Ed}}{t^2} \tag{8.15}$$

(4) At every point in the plate assembly, the design stress $\sigma_{eq,Ed}$ should satisfy the condition:

$$\sigma_{eq,Ed} \le \frac{4}{\gamma_{M0}}\frac{m_{x,Rd}}{t^2} \tag{8.16}$$

8.2.5 Design using standard elastic formulae

(1) For the design of unstiffened plates and plate assemblies of standard shape and subjected only to standard load cases, the formulae for elastic bending moments given in Annex B may be applied.

(2) The maximum von Mises equivalent bending stress should be taken for the plate geometry and loading condition using the appropriate design values of the loading and the formulae from Annex B to find $\sigma_{\mathrm{eq,Ed}}$. Bending stresses on the centreline and on the boundaries should be separately obtained, as appropriate.

(3) The equivalent bending resistance $\sigma_{\mathrm{eq,Rd}}$ should be found using Formula (8.8).

(4) It should be verified that, at each location

$$\sigma_{eq,Ed} \leq \sigma_{eq,Rd} \tag{8.17}$$

8.2.6 Design using standard plastic formulae

(1) For the design of unstiffened plates and plate assemblies of standard shape and subjected only to standard load cases, the formulae for plastic collapse given in Annex C may be applied.

(2) The design value of the reference pressure $p_{\mathrm{r,Ed}}$ should be found as

$$p_{r,Ed} = \gamma_F p_r \tag{8.18}$$

(3) The bending resistance m_{Rd}, for insertion into the formulae in Annex C, should be assessed using Formulae (8.1) and (8.5). These formulae then give the design value of the plastic collapse reference pressure $p_{\mathrm{r,Rpld}}$.

(4) It should be verified that for the plate

$$p_{r,Ed} \leq p_{r,Rpld} \tag{8.19}$$

8.2.7 Design using global computational analysis

8.2.7.1 Linear-elastic global computational analysis

(1) Where the internal stresses in a plate assembly are determined by a global computational elastic analysis, the maximum surface von Mises equivalent stress $\sigma_{\mathrm{eq,Ed}}$ at each location in each plate of the plate assembly should be calculated from the stress resultants using 8.2.2.

(2) Bending stresses within the exclusion distance $x_{\mathrm{e}} = 10t$ of the edge of a plate may be ignored in this evaluation, where t is the local plate thickness.

(3) The resistance should be verified using Formulae (8.1) to (8.13).

8.2.7.2 Nonlinear global computational analysis

(1) Where nonlinear computational analysis is used, the design plastic failure resistance shall be determined as a load factor $R_{plf,d}$ applied to the design values F_{Ed} of the combination of actions for the relevant load case.

(2) In a GMNA analysis based on the design yield strength f_y/γ_{M0} the plate assembly should be subject to the design values of the load cases detailed in (2), progressively increased by the load ratio R until the plastic failure limit condition is reached at R_{GMNA}.

(3) Where large displacements occur in the structure before a peak load is reached under GMNA, a deflection limiting criterion should applied to determine the value of R_{GMNA}. The recommended criterion is given by δ_{lim}, defined as

$$\delta_{\lim} = \frac{\Delta_{\max}}{b} \leq \frac{1}{25} \tag{8.20}$$

where

$\Delta_{\max}$ is the maximum deflection from the original plane of the plate;

b is defined in Figure 3.2 (where it is b_1 for a trapezoidal plate).

NOTE Although Formula (8.20) appears to be a serviceability restriction because it is a displacement limitation rather than a measure of structural failure, yet it is vital to have a limiting deformation as an ultimate limit criterion since both stable post-buckling deformations and unacceptably large deflections are not acceptable to the society. They are therefore treated here as an ultimate limit state. See also prEN 1993-1-6:2023, Figure 9.10.

(4) The characteristic plastic failure resistance $R_{\text{plf,k}}$ should be taken as R_{GMNA}.

(5) The design plastic failure resistance $F_{R,plf,d}$ shall be obtained from:

$$F_{R,plf,d} = \frac{F_{R,plf,k}}{\gamma_{M0}} = \frac{R_{plf,k} F_{Ed}}{\gamma_{M0}} = R_{plf,d} F_{Ed} \tag{8.21}$$

(6) It shall be verified that:

$$F_{Ed} \leq F_{R,plf,d} = R_{plf,d} F_{Ed} \text{ or } R_{plf,d} \geq 1 \tag{8.23}$$

8.3 Cyclic plasticity limit state (LS2)

8.3.1 General

(1) Repeated occurrence of strains in the plastic range in alternating directions (i.e. cyclic plasticity) may lead to ratcheting or low-cycle fatigue. Design against these limit states may be carried out by either of the following design approaches:

— When using a design method based on notional linear-elastic stresses (LA), the calculated elastic stress at any point is permitted to exceed the yield stress within the limits given in 8.3.2. The primary stresses should not exceed the yield stress. The sum of primary and secondary elastic stresses may exceed the yield stress.

— When using a design method based on the determination of the sequence, number of occurrences and accumulation of total plastic and elastic strains, nonlinear kinematic hardening laws may be considered when determining the accumulated plastic strains under repeated loading.

(2) Unless a different definition is specified, the design values of the actions for each load case should be chosen as the characteristic values of those parts of the total actions that are expected to be applied and removed more than three times in the design life of the structure.

(3) Where a materially nonlinear computational analysis is used, the varying part of the actions between the extreme upper and lower values should be considered to act in the presence of coexistent permanent parts of the load.

8.3.2 Stress-based design

(1) The plate assembly should be analysed using an LA or GNA analysis using the two extreme design values of the actions F_{Ed}. For each extreme load condition in the cyclic process, the stress components should be evaluated.

(2) From adjacent extremes in the cyclic process, the design values of the change in each stress component $\Delta\sigma_{x,Ed,i}$, $\Delta\sigma_{y,Ed,i}$, $\Delta\tau_{xy,Ed,i}$ on each plate assembly surface (represented as i=1,2 for the inner and outer surfaces of the plate assembly) and at any point in the structure should be determined. From these changes in stress, the design value of the von Mises equivalent stress change on the inner and outer surfaces should be found from:

$$\Delta\sigma_{\mathrm{eq,Ed,i}} = \sqrt{\Delta\sigma^2_{\mathrm{x,Ed,i}} - \Delta\sigma_{\mathrm{x,Ed,i}} \cdot \Delta\sigma_{\mathrm{y,Ed,i}} + \Delta\sigma^2_{\mathrm{y,Ed}} + 3\Delta\tau^2_{\mathrm{x\,y,Ed,i}}} \tag{8.24}$$

(3) The design value of the stress range $\Delta\sigma_{eq,Ed}$ should be taken as the largest change in the von Mises equivalent stress changes $\Delta\sigma_{eq,Ed,i}$, considering each plate assembly surface in turn (i = 1 and i = 2 considered separately).

(4) At a junction between plates, where the analysis models the intersection of the middle surfaces and ignores the finite size of the junction, the stress range may be taken at the first physical point in the plate assembly (as opposed to the value calculated at the intersection of the two middle surfaces).

NOTE This allowance is relevant where the stress has a steep gradient close to the junction.

(5) The design value of the stress range $\Delta\sigma_{eq,Ed}$ should be found as the largest value of the von Mises equivalent stress range in the plate or panel.

(6) The von Mises equivalent stress range should be found at every point in each plate or panel under the relevant combination of design actions as

$$\Delta\sigma_{eq,Ed} = \sqrt{\Delta\sigma^2_{\mathrm{x,Ed}} - \Delta\sigma_{x,\mathrm{Ed}}\Delta\sigma_{y,\mathrm{Ed}} + \Delta\sigma^2_{y,\mathrm{Ed}} + 3\Delta\tau^2_{\mathrm{Ed}}} \tag{8.25}$$

(7) In a materially linear design the resistance of a plate or panel against cyclic plasticity or low cycle fatigue should be verified using the von Mises equivalent stress range limitation $\Delta\sigma_{Rd}$.

$$\Delta\sigma_{Rd} = 2f_{yd} \tag{8.26}$$

in which

$$f_{yd} = f_y / \gamma_{M4} \tag{8.27}$$

(8) At every point in a plate assembly, the design stress range $\Delta\sigma_{eq,Ed}$ should satisfy the condition:

$$\Delta\sigma_{eq,Ed} \leq \Delta\sigma_{Rd} \tag{8.28}$$

NOTE This treatment aims at achieving an elastic shake-down after very few cycles of loading and is generally conservative.

8.3.3 Design using global computational analysis – accumulated strains

(1) If a materially nonlinear computational analysis is carried out, it should be undertaken using a GMNA treatment. The plate should be subject to the design values of the actions.

(2) All plastic strain increments should be taken as positive, irrespective of their direction.

NOTE Plastic strain is assessed as always positive, irrespective of its direction. Thus, a change in the direction of incremental plastic straining continues to increase the total accumulated plastic strain.

(3) The total accumulated von Mises equivalent plastic strain $\varepsilon_{eq,Ed}$ at the end of the design life of the structure should be assessed using an analysis that models all cycles of loading.

(4) Unless a more refined analysis is carried out the total accumulated von Mises equivalent plastic strain $\varepsilon_{eq,Ed}$ may be determined from:

$$\varepsilon_{eq,Ed} = n_{cyc}\,\Delta\varepsilon_{eq,Ed} \tag{8.29}$$

where

n_{cyc} is the number of cycles in the design life;

$\Delta\varepsilon_{eq,Ed}$ is the largest increment in the von Mises equivalent plastic strain during one complete load cycle at any point in the structure occurring after the third cycle.

(5) Unless a more sophisticated low cycle fatigue assessment is undertaken, the design value of the total accumulated von Mises equivalent plastic strain $\varepsilon_{eq,Ed}$ should satisfy the condition

$$\varepsilon_{p,eq,Ed} \le a_{p,eq}\left(0{,}04 - f_{yd} / 40000\right) \tag{8.30}$$

where f_{yd} is the design value of the yield stress according to 8.3.2.

The value of $a_{p,eq}$ should be taken as $a_{p,eq}$ = 2, unless relevant test data shows that a higher value is appropriate.

NOTE The partial factor on the yield stress γ_{M4} is applied for cyclic plasticity. The total acceptable plastic strain is here reduced by the factor $a_{p,eq}$ to take some account of the differences between cyclic and monotonic straining.

8.4 Buckling limit state (LS3)

8.4.1 General

(1) All relevant combinations of actions causing compressive membrane stresses or shear membrane stresses in the plate elements shall be taken into account.

(2) Where the effect could be significant, the pre-buckling out-of-plane deformations caused by transverse loads acting on the plate surface shall be considered in the design against buckling.

8.4.2 Design using buckling formulae

(1) Where an assessment of buckling of an unstiffened plate is to be undertaken using formulae, see the provisions of EN 1993-1-5.

NOTE In unstiffened plate assemblies, the condition above adjacent to local supports is the most likely location for buckling. It involves highly localized buckles in a plate where the stress distribution is varying rapidly as a combination of shear, horizontal tension and some vertical compression. For plate assemblies used for silo construction or similar structures, the provisions of EN 1993-4-1 can be used where relevant.

8.4.3 Design by global computational analysis

(1) Where a computational analysis is used for the verification of buckling (LBA, GNA, GMNA or GMNIA), attention should be paid to buckling modes involving the interaction between different plate or panels, local stress conditions, and plate boundaries that are free or only stiffened by structural members.

(2) When computational analyses are used, consideration should be given to the possible effects of imperfections.

These imperfections can be:

— geometrical imperfections:

  — global deviations on the boundaries between one plate and another;

  — deviations in any stiffening member used as a boundary condition for a plate;

    — deviations from the nominal geometric shape of each plate (initial deformation, out-of-plane deflections);

    — misalignment of butt joints;

    — deviations from nominal thickness.

— material imperfections:

    — residual stresses because of rolling, pressing, welding, straightening;

    — non-homogeneities and anisotropies.

(3) When undertaking a GMNIA computational analysis to verify buckling, the geometrical and material imperfections should be taken into account by using an initial equivalent geometric imperfection in the perfect plate or between plate or panels. The shape of the initial equivalent geometric imperfection may be derived from the relevant linear buckling mode (LBA).

(4) The amplitude of an imperfection may be taken as $e_0 = b/400$ in a plate of width *b*.

(5) The pattern of the equivalent geometric imperfections should, if relevant, be adapted to the constructional detailing and to imperfections expected from fabricating or manufacturing.

(6) In all cases, the reliability of a computational analysis should be checked using known results from tests or benchmark analysis cases (see EN 1993-1-14).

8.5 Fatigue limit state (LS4)

(1) The design stress methods identified in 7.5 should be followed.

(2) If the nominal stress method EN 1993-1-9 is applicable and used, the calculation of design stress ranges should consider the membrane stresses in the plate middle surface and the corresponding stress concentration factors k_f, where appropriate.

(3) In cases where the hot spot method of prEN 1993-1-9:2023, Annex B is applied, the fatigue action effects and the fatigue resistance should be calculated by the methods specified in that standard. Principal stress ranges at the plate surface, extrapolated to the notch location as specified in prEN 1993-1-9:2023, B.3.2(5) to (7) should be used in this case.

9 Ultimate limit state design of uni-directionally stiffened plates

9.1 General

(1) Uni-directionally stiffened plates shall be designed against the limit states LS1 to LS4 as described in this clause, using either stress-based design, design using standard formulae or global computational design.

9.2 Plastic failure limit state (LS1)

9.2.1 Stress-based design

(1) The two-dimensional stress field in the plate may be determined either by considering the orthogonal design models shown in Figure 6.7 or by more sophisticated computational calculations.

(2) The von Mises equivalent stress $\sigma_{eq,Ed}$ in the plate may be determined as follows:

$$\sigma_{eq,Ed} = \sqrt{\sigma_{x,Ed}^2 - \sigma_{x,Ed}\,\sigma_{y,Ed} + \sigma_{y,Ed}^2 + 3\tau_{xy,Ed}^2} \qquad (9.1)$$

where $\sigma_{x,Ed}$, $\sigma_{y,Ed}$ and $\tau_{xy,Ed}$ are stresses resulting from the membrane forces and bending moments in the stiffened plate

$$\sigma_{x,Ed} = \frac{n_{x,Ed}}{t} \pm \frac{4m_{x,Ed}}{t^2} \tag{9.2}$$

$$\sigma_{y,Ed} = \frac{n_{y,Ed}}{t} \pm \frac{4m_{y,Ed}}{t^2} \tag{9.3}$$

$$\tau_{xy,Ed} = \frac{n_{xy,Ed}}{t} \pm \frac{4m_{xy,Ed}}{t^2} \tag{9.4}$$

with $n_{x,Ed}$, $n_{y,Ed}$, $n_{xy,Ed}$, $m_{x,Ed}$, $m_{y,Ed}$ and $m_{xy,Ed}$ defined in 3.2 (5) and (7).

NOTE The above formulae give a simplified and generally conservative equivalent stress for design.

(3) The stress $\sigma_{x,Ed}$ in the longitudinal direction of the plate attached to the stiffener may be determined from the beam-theory internal forces N_{Ed} and $M_{x,Ed}$ in the stiffener:

$$\sigma_{x,Ed} = \frac{N_{Ed}}{A_{sl}} \pm \frac{eM_{x,Ed}}{I_{sl}} \tag{9.5}$$

where

N_{Ed}	is the axial force in the stiffener;
$M_{x,Ed}$	is the bending moment in the stiffener;
e	is the distance between the plate middle surface and the centroid of the stiffener;
A_{sl}	is the cross-sectional area of the effective stiffener section;
I_{sl}	is the second moment of area of the effective stiffener section.

(4) At each relevant section in the plate on and between the stiffeners, the design stress $\sigma_{eq,Ed}$ should satisfy the condition:

$$\sigma_{eq,Ed} \leq f_y / \gamma_{M0} \tag{9.6}$$

(5) The effect of fastener holes should be taken into account in accordance with EN 1993-1-1:2022, 8.2.3 for tension and EN 1993-1-1:2022, 8.2.4 for compression.

(6) The design of long bolted joints should account for the non-uniform distribution of forces and stresses to be transferred by the bolts, as well as through the net section of the adjacent plate, following the pertinent rules in EN 1993-1-1 and EN 1993-1-8.

9.2.2 Design using global computational analysis

9.2.2.1 Linear-elastic global computational analysis

(1) If the internal stresses in a uni-directionally stiffened plate assembly are determined by a global computational analysis based on a materially linear analysis (LA), the maximum von Mises equivalent stress $\sigma_{eq,Ed}$ in the plate assembly should be determined from the calculated stress resultants.

(2) The equivalent von Mises equivalent stress $\sigma_{eq,Ed}$ should be determined by applying Formula (9.1).

9.2.2.2 Nonlinear global computational analysis

(1) The resistance of uni-directionally stiffened plate assemblies against plastic failure may be assessed by either a materially nonlinear analysis (MNA) or a geometrically and materially nonlinear analysis (GMNA).

(2) Materially nonlinear analysis (MNA) is based on plate bending theory applied to the perfect plate structure, accounting for plasticity but with no account taken of changes of geometry (small deflection theory). The elastic–perfectly plastic material law should be assumed.

(3) Where the effects of geometric nonlinearity and/or an accurate description of the nonlinear stress-strain relationship for the material of the plate are significant, a GMNA analysis may be advantageous.

(4) The design values of the actions at the ultimate limit state should be applied to the global model of the structure.

(5) The design verification against plastic failure should satisfy Formula (9.7):

$$\varepsilon_{eq,Ed} \leq \varepsilon_{\lim} \tag{9.7}$$

in which

$$\varepsilon_{eq,Ed} = \sqrt{\varepsilon^2_{\mathrm{x},Ed} - \varepsilon_{\mathrm{x},Ed}\,\varepsilon_{\mathrm{y},Ed} + \varepsilon^2_{\mathrm{y},Ed} + 3\gamma^2_{\mathrm{xy},Ed}} \tag{9.8}$$

where

$\varepsilon_{\mathrm{eq,Ed}}$ is the von Mises equivalent plastic strain;

$\varepsilon_{\lim}$ is the limiting strain for uniaxial tension at the ultimate limit state.

NOTE 1 The value of $\varepsilon_{\lim}$ = 5 % for all steel grades identified in EN 1993-1-1 and EN 1993-1-4, unless the National Annex gives a different value.

NOTE 2 The calculated value of the accumulated plastic strain is sensitive to the analysis parameters (notably the finite element type and the mesh refinement). EN 1993-1-14 addresses this issue.

(6) Where large displacements occur in the structure before a peak load is reached under GMNA, the deflection limiting criterion should be used to determine the value of R_{GMNA}. This criterion is given by $w_{\lim}$, defined as

$$w_{\lim} = \min\left(\frac{b}{25}, 20t\right) \tag{9.9}$$

where

b is as defined in Figures 3.2 for an individual plate or sub-panel;

9.3 Cyclic plasticity limit state (LS2)

(1) Repeated occurrence of strains in the plastic range (i.e. cyclic plasticity) may lead to ratcheting and low-cycle fatigue. Design against these limit states may be carried out according to the procedure in 8.3.

9.4 Buckling limit state (LS3)

9.4.1 General

(1) All relevant combinations of actions causing compressive membrane stresses or shear membrane stresses in the plate elements shall be taken into account.

(2) Where the effect could be significant, the pre-buckling out-of-plane deformations caused by transverse loads acting on the plate surface should be considered in the design against buckling.

(3) Where a uni-directionally stiffened plate or panel is loaded by in-plane compression or shear, its resistance to overall plate buckling should be verified using the design rules given in EN 1993-1-5.

(4) Flexural, lateral torsional or distortional stability of the stiffeners should be verified either using the buckling formulae defined in 9.4.2, or by nonlinear computational analysis including geometric and material nonlinearities and imperfections as defined in 9.4.3.

9.4.2 Design of the stiffener and adjacent plate using buckling formulae

(1) If the simplified analysis method of 6.5.3 is used, the buckling verification of a stiffener i of the uni-directionally stiffened plate or panel may be performed by considering the stiffener and an effective width of the plate as an isolated member of cross-sectional area A_i and using the interaction formulae in EN 1993-1-1:2022, 8.3.3 together with the additional rules in EN 1993-1-1:2022, Annex C for mono-symmetric sections, and taking the following loading conditions into account:

— effects of out-of-plane loads;

— equivalent axial force in the effective cross section due to longitudinal membrane stresses in the plate;

— eccentricity e of the equivalent axial force N_{Ed} relative to the centroid of the effective cross-section.

(2) The effective area A_i of the stiffener and plate parts considered as isolated members should be calculated by considering the effective widths given in 6.5.3 (3) or EN 1993-1-5.

(3) The effective second moment of area of the combined stiffener and plate should be determined considering the same effective widths for the plate parts adjacent to the stiffener as those used in the calculation of A_i.

(4) In addition to the buckling check using formulae, both sides of the cross-section at the ends and at the midpoint of the stiffener should be checked for LS1. In-plane shear shall be included in the check if transverse loads are significant.

9.4.3 Design of the stiffener and adjacent plate using computational analysis

(1) If the plate buckling resistance for combined in-plane and out-of-plane loads is checked by a computational analysis, the design value of the actions F_{Ed} should satisfy the condition:

$$F_{Ed} \leq F_{Rd} \tag{9.16}$$

(2) The plate buckling resistance F_{Rd} of the plate assembly is defined as:

$$F_{Rd} = k\, F_{Rk}/\gamma_{M1} \tag{9.17}$$

where

F_{Rk} is the characteristic buckling resistance of the plate assembly;

k is the calibration factor (see (6));

(3) The characteristic buckling resistance F_{Rk} should be derived from a load-deformation curve which is calculated for the relevant point of the structure when subject to the relevant combination of design actions F_{Ed}. The analysis should account for the imperfections of the plate between stiffeners as indicated in Figure 9.1 with the amplitude e_0 defined in Formula (9.18) and the global stiffener imperfection amplitude defined in prEN 1993-1-5:2022, Annex C.

$$e_0 = \frac{b}{400} \tag{9.18}$$

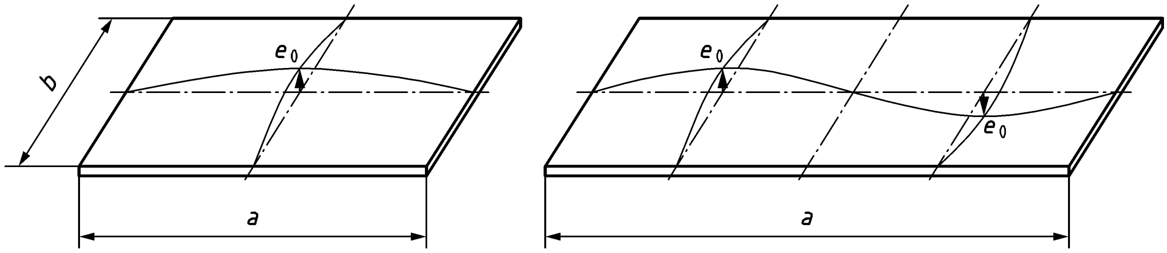


Figure 9.1 — Equivalent geometric imperfection in a plate between stiffeners, with one half-wave in a short plate and two or more half-waves in longer plates

NOTE The value of e_0 has been chosen to be compatible with EN 1993-1-5 and is a smaller value than that defined in EN 1090-2.

(4) The characteristic buckling resistance F_{Rk} is defined by one of the two following criteria:

— maximum load in the load-deformation-curve (limit load);

— maximum tolerable deformation before reaching a bifurcation load or a limit load, if relevant.

— The limit strain $\varepsilon_{\mathrm{lim}}$ defined in 9.2.2.2 should not be exceeded at any point in the stiffened plate assembly.

(5) The verification of the critical buckling resistance obtained from a computational analysis should be checked by calculating other plate buckling cases for which characteristic buckling resistance values $F_{Rk,known}$ are known, with the same or essentially similar imperfection assumptions. These check cases should be similar in their buckling controlling parameters (non-dimensional plate slenderness, post-buckling behaviour, imperfection-sensitivity and material behaviour).

(6) Depending on the results of the verification checks a calibration factor k should be evaluated from:

$$k = F_{Rk,known} / F_{Rk,check} \tag{9.19}$$

where

$F_{\mathrm{Rk,known}}$ is derived from existing reliable information;

$F_{\mathrm{Rk.check}}$ are the results of the computational calculations.

9.5 Fatigue limit state (LS4)

(1) The design stress methods identified in 7.5 should be followed.

(2) If the nominal stress method EN 1993-1-9 is applicable and used, the calculation of design stress ranges should consider the membrane stresses in the plate middle surface and the corresponding stress concentration factors k_{f}, where appropriate.

(3) In cases where the hot spot method of Annex B is applied, the fatigue action effects and the fatigue resistance should be calculated by the methods specified in prEN 1993-1-9:2023, Annex B. Principal stress ranges at the plate surface, extrapolated to the notch location, as specified in prEN 1993-1-9:2023, B.3.2(5) to (7) should be used in this case.

10 Ultimate limit state design of bi-directionally stiffened plates

10.1 General

(1) The rules of this standard apply only to rectangular plates with stiffeners parallel to and orthogonal to the long dimension of the plate. Both sets of stiffeners are assumed to be continuous (i.e. not staggered).

(2) Orthogonally stiffened plates shall be designed against the limit states LS1 to LS4. If not specifically modified in this sub-clause, the pertinent rules from Clauses 7 to 9 apply also to bi-directionally stiffened plates.

(3) These limit states may be deemed to be satisfied if the provisions of this sub-clause is used, either by stress-based design or by computational design.

(4) If a stiffened plate or panel is modelled as a grillage as described in 6.5.4, the cross-section resistance and the buckling resistance of the individual members i of the grillage should be checked for the combination of in-plane and out-of-plane loading effects using the interaction formula in EN 1993-1-1:2022, 8.3.3.

(5) In determining the cross-sectional area A_i of the effective plate of an individual member i of the grillage, the effects of shear lag should be taken into account using the reduction factor β according to EN 1993-1-5.

(6) For a member of the grillage (Figure 6.8) that is arranged in parallel to the direction of in-plane compression forces, the cross-sectional area should be determined taking account of the effective width of the adjacent sub-panels due to plate buckling according to EN 1993-1-5.

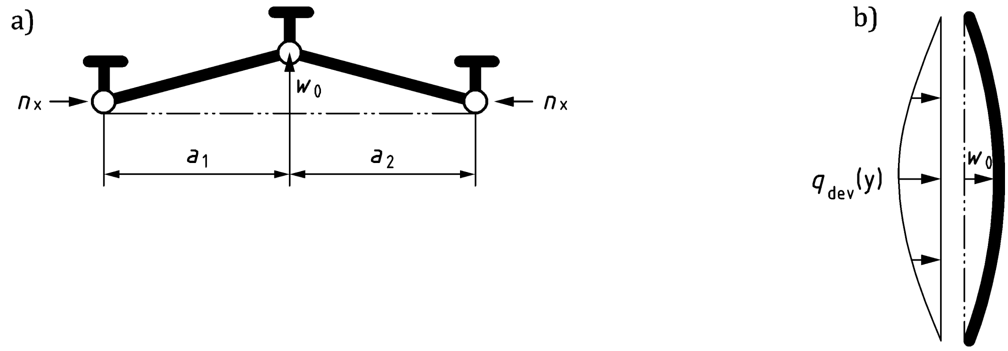
10.2 Stress-based design

(1) The two-dimensional stress field in the bi-directionally stiffened plate may be determined either by considering a grillage model or by use of a more sophisticated computational treatment.

(2) If a stiffened plate or panel is modelled as a grillage as described in 6.5.4, the cross-section resistance in the individual members of the grillage (Figure 6.8) should be checked for the combination of in-plane and out-of-plane loading effects in accordance with EN 1993-1-1, treating each member of the grillage as an individual member. For members in compression, the buckling resistance is checked by including second order internal forces in the cross-sectional verification, see (5) to (7).

(3) In determining the cross-sectional area A_i of the effective plate of an individual member i of the grillage, the effective width should be found according to 6.5.4 (2).

(4) The buckling check of the individual member "i" may be performed by treating the stiffener and effective plate as a simply supported member subject to lateral loading with an initial sinusoidal imperfection w_0 equal to $s/200$, where s is the smallest of a_1, a_2 or b (Figures 6.8 and 10.1). The eccentricity of any stiffener should be included in the check.



a) Section through three transverse stiffeners half-way between longitudinal stiffeners

b) Section showing assumed deviation of the central transverse stiffener in a)

Figure 10.1 — Deviation forces and imperfections in a transverse stiffener “j"

(5) Each member “i” should by designed for the forces that arise from the imperfection deviation w_0 coupled with the compression in adjacent sub-panels and transversely orientated stiffeners of the grillage. The latter should be determined using the assumption that the two adjacent stiffeners in the orthogonal orientation are rigid and that the compressed sub-panels and the transversally orientated stiffeners between these stiffeners are simply supported (Figure 10.1 a).

(6) In addition to the cross-sectional check with second order internal forces, it should be checked that the additional deflection in the member in a second order elastic analysis that stem from the effects in (5) does not exceed $b/200$.

(7) The sub-panels may be designed for LS1 to LS4 applying the methods in clause 8. The stress state may be determined by superposition of the stresses from the grillage model with stresses in the sub-panel determined in accordance with Annex B.

(8) Plastic reference resistances in accordance with Annex C may only be applied to the design of individual sub-panels if the design values of the membrane compressive stresses in the plate due to the bending of the stiffeners or due to external forces in the plane of the plate do not exceed 15 % of the design value of the yield stress f_y/γ_{M0}.

10.3 Design using computational analysis

(1) Bi-directionally stiffened plates may be designed by computational analysis. The pertinent rules in Clauses 8 and 9 apply accordingly.

Annex A
(informative)

Membrane and simple elastic bending analysis stress resultants in plates and plate assemblies

A.1 Use of this Annex

(1) This Informative Annex provides design formulae for the simple calculation of bending moments and membrane forces in rectangular, trapezoidal and triangular plate assemblies.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

A.2 Scope and field of application

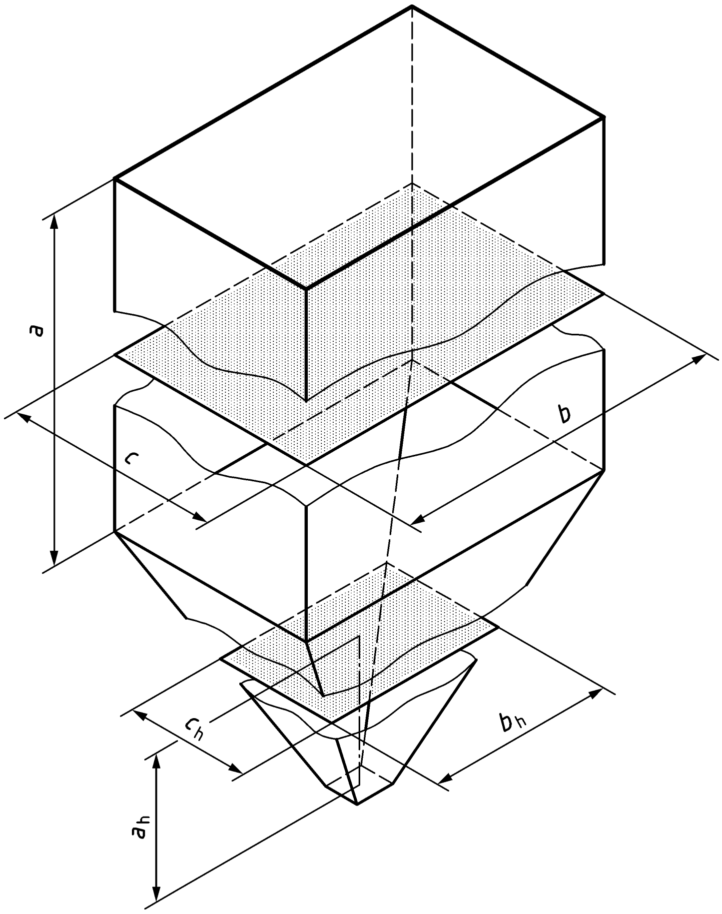
(1) This Informative Annex provides design formulae for the simple calculation of bending moments and membrane forces in rectangular, trapezoidal and triangular plate assemblies by treating individual horizontal slices of the plate walls (Figure 6.1) as horizontal plane frames and using simple analyses.

(2) This Informative Annex provides design formulae for the following load cases:

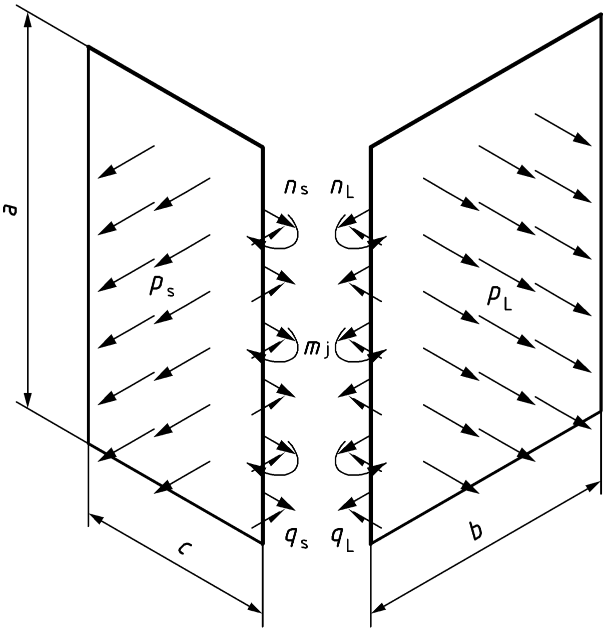
— rectangular box assemblies under uniformly distributed pressure, see A.5;

— trapezoidal and triangular plate assemblies under uniformly distributed pressure, see A.6.

(3) The doubly-symmetric form of plate assembly treated by this annex is illustrated in Figure A.1.



a) Characteristic plate assembly forming a box structure



b) Plate to plate interactions and loading on individual plates

Figure A.1 — Type of plate assembly considered in this annex

A.3 Symbols

(1) The symbols relating to a horizontal slice of the structure, as shown in Figures 6.3, 6.4 and A.1, are defined follows:

a height of a vertical plate in the assembly;

a_h height of an inclined trapezoidal plate in the hopper of an assembly;

b length of the longer side of a vertical plate assembly;

b_h length of a horizontal section at any level of the longer side within a trapezoidal plate;

c length of the shorter side of a vertical plate assembly;

c_h length of a horizontal section at any level of the shorter side within a trapezoidal plate;

n_L horizontal membrane tension per unit width in the long side (L) of a rectangular plate assembly;

n_S horizontal membrane tension per unit width in the short side (S) of a rectangular plate assembly;

m_J bending moment per unit width between the shorter and longer sides of a rectangular plate assembly;

p_L horizontally uniform pressure acting on the longer side at any level;

p_S horizontally uniform pressure acting on the shorter side at any level;

q_L horizontal transverse shear per unit width on the edge of the long plate in a rectangular plate assembly;

q_S horizontal transverse shear per unit width on the edge of the short plate in a rectangular plate assembly;

t uniform thickness of a plate.

NOTE The values of pressures and stresses defined here are independent of their role as characteristic or design values. These distinctions are used in the body of the standard and can be used as appropriate by the designer.

A.4 Simplified treatment

(1) In this simplified treatment, each slice in the box structure is treated as independent of the remainder, so that a horizontal slice of the structure may be assumed to develop bending moments about a vertical axis (inducing horizontal bending stresses) and horizontal membrane forces associated only with the pressures acting at that level.

(2) The interactions between the plate or panels of the vertical part of a plate assembly are illustrated in Figure 6.3.

(3) In the vertical or meridional direction, additional bending moments about a horizontal axis (inducing vertical bending stresses) and vertical or meridional membrane forces develop, associated with the pressure variation down that meridian.

(4) These more complex interactions between the plate or panels of a pyramidal hopper are illustrated in Figure 6.4.

A.5 Simple formulae for SMBT treatment of a rectangular plate assembly

(1) The development of membrane tension in each side plate of a plate assembly at a given level in the box plate assembly is illustrated in Figure 6.3. At this level, a horizontal slice is treated as acting independently of all other parts of the structure.

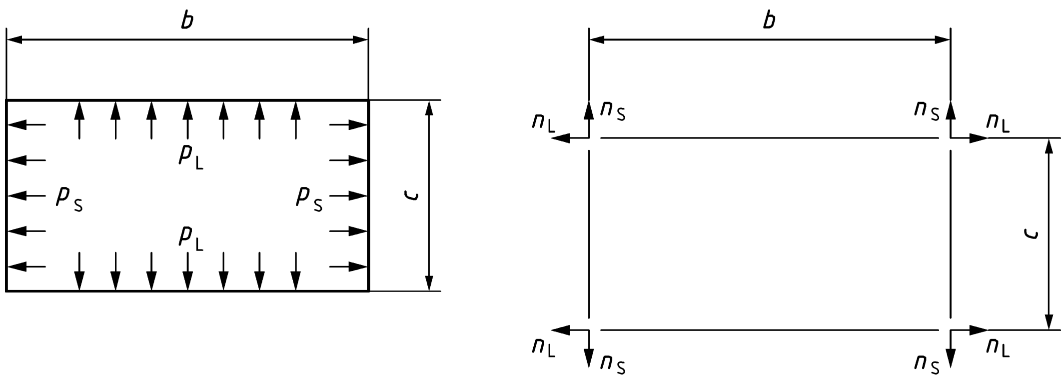
NOTE This treatment refers to the directions horizontal and vertical with reference to Figure A.1 (a) and Figure 6.1. The reason for this choice of terminology is that containment structures have pressures that vary with depth, but are largely constant at any horizontal level. Where this treatment is needed for structures with a different orientation (e.g. a horizontal air duct), the reader is requested to invert the terminology and recognize that the term "horizontal" here means "perpendicular to the direction of the structure's principal axis".

(2) In this slice of structure (Figure A.2), the pressure p_L acting on the long side b induces transverse shears at its ends which produce membrane tensile stress resultants per unit height in the short side c of magnitude:

$$n_S = \frac{p_L b}{2} \tag{A.1}$$

(3) The pressure p_S acting on the short side c induces transverse shears at its ends which produce membrane tensile stress resultants per unit height in the long side b of magnitude:

$$n_L = \frac{p_S c}{2} \tag{A.2}$$



a) Pressures at one level

b) Membrane stress resultants in plate elements

Figure A.2 — Simple treatment of membrane stress resultants at each level

(4) At the junction between the two sides, the bending moment per unit height is given by:

$$m_{junct} = -\left(\frac{p_L b^2 + k_s p_S c^2}{12\left(1 + k_s\right)}\right) \tag{A.3}$$

$$k_s = \left(\frac{c}{b}\right)\left(\frac{I_L}{I_S}\right) \tag{A.4}$$

where

I_L is the second moment of area per unit width of the long side plate;

I_S is the second moment of area per unit width of the short side plate.

NOTE 1 This formulation covers corrugated wall construction with stiff corner junctions as well as plates of uniform thickness.

NOTE 2 Formulae for the second moment of area of corrugated plates are given in EN 1993-4-1.

(5) For uniform thickness plates, the second moment of area per unit width is given by:

$$I_L = \frac{t_L^3}{12\left(1-\nu^2\right)} \text{ and } I_S = \frac{t_S^3}{12\left(1-\nu^2\right)} \quad (A.5)$$

(6) The pressures p_L and p_S induce bending moments along the long side, as shown in Figure A.3 as:

$$m_{L,mid} = m_{junct} + \frac{p_L b^2}{8} \quad (A.6)$$

and bending moments along the short side as:

$$m_{S,mid} = m_{junct} + \frac{p_S c^2}{8} \quad (A.7)$$

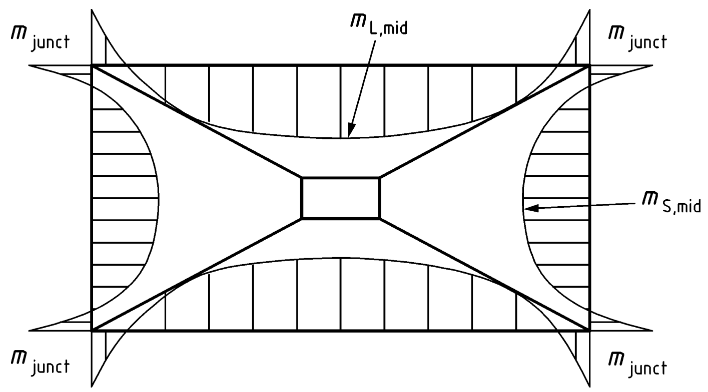


Figure A.3 — Simple treatment (not to scale) of bending moments at each level

A.6 Simple formulae for SMBT treatment of pyramidal hopper plate assemblies

(1) A vertical section intersecting the short sides of a pyramidal hopper with its inclination to the vertical of β_S is shown in Figure A.4. The section corresponding to a horizontal slice at a specific level is also indicated.

(2) The normal pressure p_n and frictional traction p_t against the hopper wall both vary with the level. On the short wall these are denoted as $p_{n,S}$ and $p_{t,S}$, whilst on the long wall they are as $p_{n,L}$ and $p_{t,L}$, this notation being compatible with EN 1991-4.

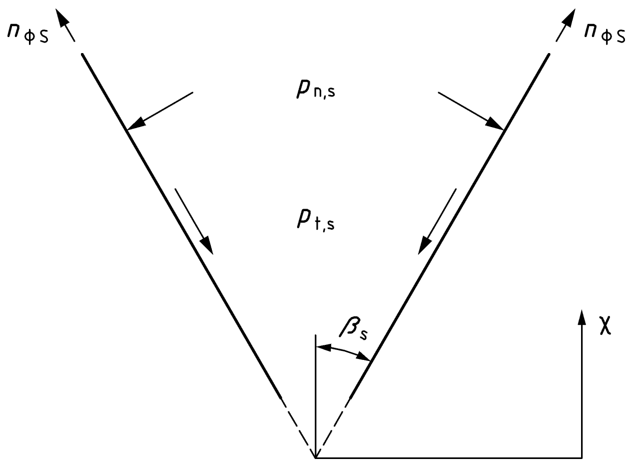


Figure A.4 — Section through short walls of a hopper with trapezoidal plates

(3) The pressure normal to the wall may be taken from EN 1991-4 as p_n normal to the inclined wall and p_t the traction down the sloping hopper wall

(4) The horizontal membrane stress resultant n_h per unit width of hopper plate in the horizontal slice are shown in Figure A.5, and may be assessed as:

$$n_{L,h} = x\left(\frac{\sin\beta_L}{\cos\beta_S}\right)\left(p_{n,S}\cos\beta_S - p_{t,S}\sin\beta_S\right) \tag{A.8}$$

$$n_{S,h} = x\left(\frac{\sin\beta_S}{\cos\beta_L}\right)\left(p_{n,L}\cos\beta_L - p_{t,L}\sin\beta_L\right) \tag{A.9}$$

where

β_L	is the hopper apex half angle for the long hopper wall;
β_S	is the hopper apex half angle for the short hopper wall.

(5) The meridional membrane stress resultant per unit width of hopper plate at the top of the hopper may be assessed for design purposes as:

$$n_{\phi,L} = k_u \frac{c p_{vft}}{2}\left(\frac{b}{b+c}\right)^2 \tag{A.10}$$

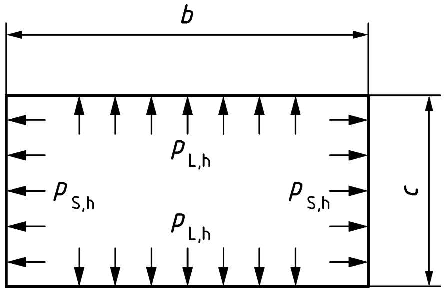
$$n_{\phi,L} = k_u \frac{b p_{vft}}{2}\left(\frac{c}{b+c}\right)^2 \tag{A.11}$$

where

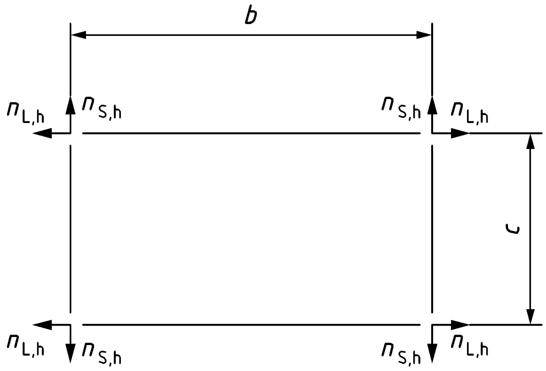
p_{vft}	is the mean vertical stress in the stored solid at the transition (see EN 1991-4);
b	is the length of the long side of the vertical section;
c	is the length of the short side of the vertical section;
k_u	is a factor to account for the non-uniformity of the meridional force.

(6) The recommended value of k_u is 2,0.

NOTE The factor k_u is used to account for the much higher meridional force at the centre of a hopper wall, due to the greater length of inclined wall subject to wall friction beneath the centre.



a) Pressures at a specific level



b) Membrane stress resultants in plate elements

Figure A.5 — Simple treatment of membrane stress resultants at each level

Annex B
(informative)

Formulae for linear elastic stresses in unstiffened rectangular plates from small deflection theory

B.1 Use of this Annex

(1) This Informative Annex provides design formulae for the calculation of internal stresses in unstiffened rectangular, trapezoidal and triangular plates.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

B.2 Scope and field of application

(1) This Informative Annex provides design formulae for the calculation of internal stresses in unstiffened rectangular, trapezoidal and triangular plates based on the small deflection theory for plates. The effects of membrane forces are not taken into account in the design formulae given in this annex. Where membrane forces induce in-plane stresses, these may be added to the bending stresses using superposition provided that the stress components in specific directions are added.

NOTE von Mises equivalent stresses cannot be added by simple superposition since the orientation of the principal membrane stresses and the principal bending stresses will not, in general, coincide.

(2) This Informative Annex provides design formulae for the following load cases:

— rectangular plates under uniformly distributed pressure, see B.5;

— trapezoidal and triangular plates under uniformly distributed pressure, see B.6;

— rectangular plates under pressure varying linearly from the top to the bottom, see B.7.

(3) The data given for each case varies, with more information for the commonest cases and only limited information for the less common ones. The focus is on the maximum bending stresses σ_{bx} and σ_{by} and their conservative combination into von Mises equivalent surface stresses σ_{bVM} using approximate formulae for each case with a defined range of validity.

NOTE 1 The values given here are based on a Poisson's ratio ν of 0,30.

NOTE 2 The stresses are combined in a conservative manner in the sense that the maximum stresses in the x and y directions are not always coincident, but are here combined to provide a safe result.

NOTE 3 Local very high stresses are predicted to occur in the corners of rectangular and trapezoidal plates and at the apex of triangular plates. These are ignored in the following data, as they are deemed irrelevant to the plastic failure limit state.

B.3 Symbols

(1) The symbols used are:

a	longer side of a rectangular plate or the height of a triangular or trapezoidal plate (Figure 3.2);
b	shorter side of a rectangular or the base of a triangular plate;
b_1	base or longer parallel side of a trapezoidal plate;
b_2	top or shorter parallel side of a trapezoidal plate;
k_x	coefficient for the maximum bending stress in the x direction σ_{bx};
k_y	coefficient for the maximum bending stress in the y direction σ_{by};
k_{VM}	coefficient for the maximum von Mises equivalent bending stress σ_{bVM};
p_r	local reference pressure that is used to characterise the pressure distribution;
t	uniform thickness of a plate;
ψ	aspect ratio of a plate ($\psi = b/a$).

NOTE The values of pressures and stresses defined here are independent of their role as characteristic or design values. These distinctions are used in the body of the standard and can be used as appropriate by the designer.

B.4 Characterization of stresses

(1) The maximum bending stresses σ_{bx}, σ_{by} and σ_{bVM} in each plate may be determined using the following formulae:

$$\sigma_{bx} = k_x p_r \left(\frac{b}{t}\right)^2 \quad \text{(B.1)}$$

$$\sigma_{by} = k_y p_r \left(\frac{b}{t}\right)^2 \quad \text{(B.2)}$$

$$\sigma_{bVM} = k_{VM} p_r \left(\frac{b}{t}\right)^2 \quad \text{(B.3)}$$

where

p_r is the reference pressure for the distribution acting on the plate.

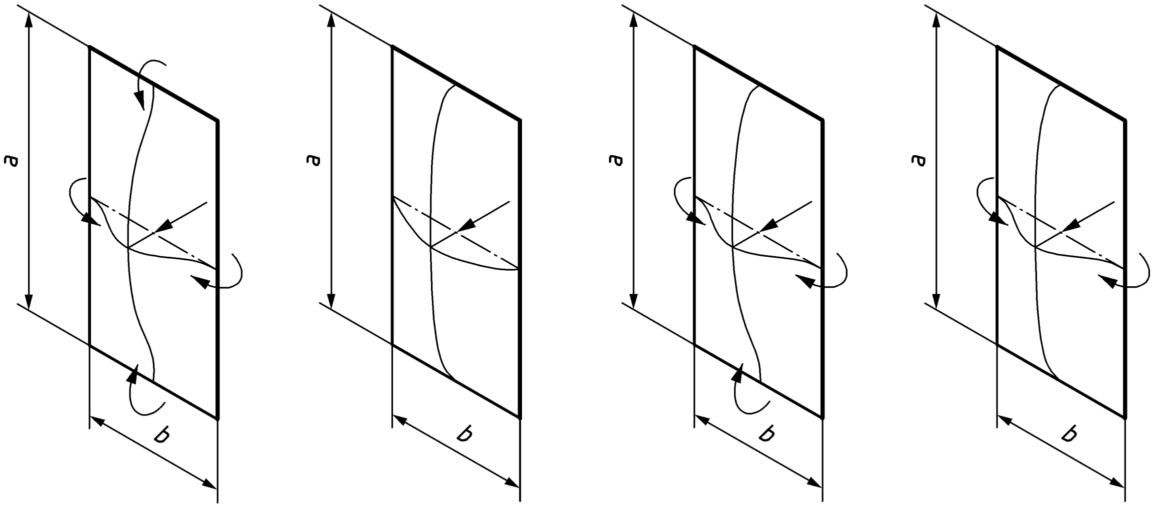
For uniform pressure p_r is the uniform value: for other patterns of loading, the value of p_r is defined for the particular load case.

(2) The von Mises equivalent stress has been assessed using the maximum bending stresses in the two different principal directions. In some cases these may not be coincident, so this process may slightly overestimate the true value, making it a conservative choice.

$$\sigma_{bVM} = \sqrt{\sigma_{bx}^2 + \sigma_{by}^2 - \sigma_{bx}\,\sigma_{by}} \quad \text{(B.4)}$$

NOTE 1 The points for which the state of stress is defined in the data tables are located either on the centre lines or on the boundaries. Due to symmetry in the assumed boundary conditions, the twisting stresses τ_b are generally very low and can be ignored.

NOTE 2 Maximum stresses in cases with asymmetric loading or boundary conditions will not necessarily occur at mid-point of edges or centrelines. For plates simply-supported all round (boundary condition SCB, see Fig. B.1, the maximum y-direction stresses that are given can also not occur at $y = a / 2$).



a) Support condition: SCA FFFF b) Support condition: SCB SSSS c) Support condition: SCC FFFS d) Support condition: SCD SFFS

Figure B.1 — Support conditions with load direction and hogging moment directions

B.5 Rectangular plates under uniform pressure

B.5.1 Boundary conditions

(1) The boundary conditions used here all involve complete restraint against transverse displacements on all edges.

(2) The different defined boundary conditions according to Table 6.4 are used, but with two equivalences for additional ease of use:

— BC2r: rotationally restrained edge is also termed F (fixed);

— BC2f: rotationally free edge is also termed S (simply supported).

(3) For this load case in rectangular plates, the following support conditions (SC) are defined for four boundary condition combinations with the notation F and S in the sequence: base, two sides, top according to Figure B.1:

— SCA: All sides rotationally restrained: BC2r (FFFF);

— SCB: All sides simply supported: BC2f (SSSS);

— SCC: Base and sides rotationally restrained, top simply supported: BC2r and BC2f (FFFS);

— SCD: Base and top simply supported, sides rotationally restrained: BC2r and BC2f (SFFS).

B.5.2 Stress descriptors

(1) For this load case, the notation k_{xU}, k_{yU} and k_{VMU} is used to identify the uniform loading U on a rectangular plate.

(2) The load case is illustrated in Figure B.2.

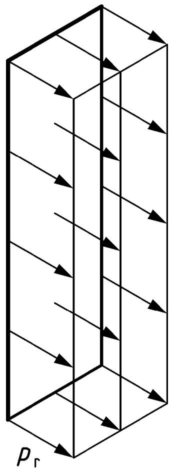


Figure B.2 — Uniform pressure on a rectangular plate with reference pressure p_r

B.5.3 Stresses for SCA: FFFF

(1) The moment stress coefficients for SCA are given in Table B.1.

Table B.1 — Moment stress coefficients for boundaries SCA

Moment location	Moment stress coefficient	Value for ψ < 0,33
von Mises stress at centre	$k_{VMU} = -0{,}166\psi^2 + 0{,}101\psi + 0{,}259$	$k_{VMu} = 0{,}274$
von Mises stress on side	$k_{VMU} = -0{,}452\psi^2 + 0{,}289\psi + 0{,}481$	$k_{VMU} = 0{,}524$
von Mises stress on base	$k_{UVMU} = -0{,}165\psi^2 + 0{,}172\psi + 0{,}315$	$k_{VMU} = 0{,}356$
max sagging x moment on symmetry axis	$k_{xU} = -0{,}233\psi^2 + 0{,}134\psi + 0{,}235$	$k_{xU} = 0{,}252$
max sagging y moment on symmetry axis	$k_{yU} = -0{,}509\psi^3 + 1{,}02\psi^2 - 0{,}579\psi + 0{,}206$	$k_{yU} = 0{,}107$
most negative hogging y moment on base	$k_{yU} = 0{,}158\psi^2 - 0{,}166\psi - 0{,}301$	$k_{yU} = -0{,}342$
most negative hogging x moment on side	$k_{xU} = 0{,}433\psi^2 - 0{,}277\psi - 0{,}461$	$k_{xU} = -0{,}502$

B.5.4 Stresses for SCB: SSSS

(1) The moment stress coefficients for SCB are given in Table B.2.

Table B.2 — Moment stress coefficients for boundaries SCB

Moment location	**Moment stress coefficient**	**Value for ψ < 0,33**
von Mises stress at centre	$k_{VMU} = 0{,}085\psi^2 - 0{,}65\psi + 0{,}97$	as per equation
von Mises stress on side	$k_{VMU} = -0{,}094\psi^2 + 0{,}137\psi$	$k_{VMU} = 0$
von Mises stress on base	$k_{VMU} = -0{,}004\psi^2 + 0{,}003\psi$	$k_{VMU} = 0$
max sagging x moment on symmetry axis	$k_{xU} = 0{,}174\psi^2 - 0{,}884\psi + 1$	as per equation
max sagging y moment on symmetry axis	$k_{yU} = -0{,}165\psi^2 + 0{,}257\psi + 0{,}2$	$k_{yU} = 0{,}27$
most negative hogging y moment on base	$k_{yU} = 0$	$k_{yU} = 0$
most negative hogging x moment on side	$k_{xU} = 0$	$k_{xU} = 0$

B.5.5 Stresses for SCC: FFFS

(1) The moment stress coefficients for SCC are given in Table B.3.

Table B.3 — Moment stress coefficients for boundaries SCC

Moment location	**Moment stress coefficient**	**Value for ψ < 0,33**
von Mises stress at centre	$k_{VMU} = -0{,}122\psi^2 + 0{,}080\psi + 0{,}267$	$k_{VMU} = 0{,}279$
von Mises stress on side	$k_{VMU} = -0{,}392\psi^2 + 0{,}308\psi + 0{,}467$	$k_{VMU} = 0{,}525$
von Mises stress on base	$k_{VMU} = -0{,}072\psi^2 + 0{,}082\psi + 0{,}335$	$k_{VMU} = 0{,}356$
max sagging x moment on symmetry axis	$k_{xU} = -0{,}207\psi^2 + 0{,}152\psi + 0{,}225$	$k_{xU} = 0{,}252$
max sagging y moment on symmetry axis	$k_{yU} = -0{,}103\psi^3 + 0{,}277\psi^2 - 0{,}181\psi + 0{,}154$	$k_{yU} = 0{,}121$
most negative hogging y moment on base	$k_{yU} = 0{,}069\psi^2 - 0{,}079\psi - 0{,}321$	$k_{yU} = -0{,}340$
most negative hogging x moment on side	$k_{xU} = 0{,}376\psi^2 - 0{,}295\psi - 0{,}447$	$k_{xU} = -0{,}502$

B.5.6 Stresses for SCD: SFFS

(1) The moment stress coefficients for SCD are given in Table B.4.

Table B.4 — Moment stress coefficients for boundaries SCD

Moment location	Moment stress coefficient	Value for ψ < 0,33
von Mises stress at centre	$k_{VMU} = -0{,}084\psi^2 + 0{,}063\psi + 0{,}269$	$k_{VMU} = 0{,}279$
von Mises stress on side	$k_{VMU} = -0{,}297\psi^2 + 0{,}265\psi + 0{,}469$	$k_{VMU} = 0{,}524$
von Mises stress on base	$k_{VMU} = -0{,}001\psi^2 + 0{,}001\psi + 0{,}003$	$k_{VMU} = 0{,}003$
max sagging x moment on symmetry axis	$k_{xU} = -0{,}161\psi^2 + 0{,}136\psi + 0{,}225$	$k_{xU} = 0{,}252$
max sagging y moment on symmetry axis	$k_{yU} = 0{,}093\psi^3 - 0{,}090\psi^2 + 0{,}024\psi + 0{,}120$	$k_{yU} = 0{,}121$
most negative hogging y moment on base	$k_{yU} = 0$	$k_{yU} = 0$
most negative hogging x moment on side	$k_{xU} = 0{,}284\psi^2 - 0{,}254\psi - 0{,}449$	$k_{xU} = -0{,}502$

B.6 Trapezoidal and triangular plates under uniformly distributed pressure

B.6.1 General

(1) The additional notation $\psi_1 = b_1/a$ is used for trapezoidal and triangular plates.

(2) The range of geometries covered by this annex is $0{,}50 \leq b_1/a = \psi_1 \leq 3$.

(3) Where the trapezoidal plate reaches the dimensions $b_1 = b_2$, the formulae given here match those for the rectangular plate with the same boundary conditions.

B.6.2 Boundary conditions

(1) The boundary conditions used here all involve complete restraint against transverse displacements on all edges.

(2) The different defined boundary conditions according to Table 6.4 are used, but with two equivalences for additional ease of use:

— BC2r: rotationally restrained edge is also termed F (fixed)

— BC2f: rotationally free edge is also termed S (simply supported)

(3) For this load case in triangular and trapezoidal plates, the following support conditions (SC) are defined for two boundary condition combinations:

— SCA: All sides rotationally restrained: BC2r (FFFF)

— SCB: All sides simply supported: BC2f (SSSS)

B.6.3 Stress descriptors

(1) For this load case, the notation k_{xT}, k_{yT} and k_{VMT} is used to identify the uniform loading on a triangular or trapezoidal plate.

B.6.4 Stresses for SCA: FFFF

(1) The maximum bending stresses σ_{bVM} in the plate may be determined using:

$$\sigma_{VMT} = f_{VM}\, p_r \left(\frac{b_1}{t}\right)^2 \tag{B.5}$$

in which p_r is the uniform pressure acting on the plate and f_{VM} takes the value f_{VMC} on the centre line and f_{VME} at the side edge.

(2) The centreline sagging moment stress coefficients for SCA are given by:

$$f_{VMC} = c_0 + c_1\left(\frac{b_2}{b_1}\right) + c_2\left(\frac{b_2}{b_1}\right)^2 \tag{B.6}$$

$$c_0 = 0{,}159 - 0{,}122\left(\frac{b_1}{a}\right) + 0{,}025\left(\frac{b_1}{a}\right)^2 \tag{B.7}$$

$$c_1 = -0{,}168 + 0{,}237\left(\frac{b_1}{a}\right) - 0{,}058\left(\frac{b_1}{a}\right)^2 \tag{B.8}$$

$$c_2 = 0{,}413 - 0{,}395\left(\frac{b_1}{a}\right) + 0{,}085\left(\frac{b_1}{a}\right)^2 \tag{B.9}$$

(3) The edge sagging moment stress coefficients for SCA are given by:

$$f_{VME} = e_0 + e_1\left(\frac{b_2}{b_1}\right) + e_2\left(\frac{b_2}{b_1}\right)^2 \tag{B.10}$$

$$e_0 = 0{,}271 - 0{,}197\left(\frac{b_1}{a}\right) + 0{,}039\left(\frac{b_1}{a}\right)^2 \tag{B.11}$$

$$e_1 = -0{,}052 + 0{,}292\left(\frac{b_1}{a}\right) + 0{,}086\left(\frac{b_1}{a}\right)^2 \tag{B.12}$$

$$e_2 = 0{,}555 - 0{,}661\left(\frac{b_1}{a}\right) + 0{,}156\left(\frac{b_1}{a}\right)^2 \tag{B.13}$$

B.6.5 Stresses for SCB: SSSS

(1) The von Mises equivalent moment stress on the centreline of the plate for SCB are given by the following formulae:

$$\sigma_{VMT} = f_{VMT} p_r \left(\frac{b_1}{t}\right)^2 \tag{B.14}$$

$$f_{VMT} = c_0 + c_1\left(\frac{b_2}{b_1}\right) + c_2\left(\frac{b_2}{b_1}\right)^2 \tag{B.15}$$

$$c_0 = 0,08 + 0,48e^{-2,3\psi_1} \tag{B.16}$$

$$c_1 = 0,217 - 0,101\psi_1 \tag{B.17}$$

$$c_2 = 0,10 \tag{B.18}$$

B.7 Rectangular plates under linearly varying pressure from the top to the bottom

B.7.1 General

(1) The formulae given here are applicable to plates under pressure that is constant in the short (horizontal) direction and varies linearly in the long direction (hydrostatic loading).

B.7.2 Boundary conditions

(1) The boundary conditions used here all involve complete restraint against transverse displacements on all edges.

(2) The different defined boundary conditions according to Table 6.4 are used, but with two equivalences for additional ease of use with the notation F and S in the sequence: base, sides, top:

— BC2r: rotationally restrained edge is also termed F (fixed)

— BC2f: rotationally free edge is also termed S (simply supported)

(3) For this load case in rectangular plates, the following support conditions (SC) are defined for four boundary condition combinations according to Figure B.1:

— SCA: All sides rotationally restrained: BC2r (FFFF)

— SCB: All sides simply supported: BC2f (SSSS)

— SCC: Base and sides rotationally restrained, top simply supported: BC2r and BC2f (FFFS)

— SCD: Base and top simply supported, sides rotationally restrained: BC2r and BC2f (SFFS)

B.7.3 Stress descriptors

(1) For this load case, the notation k_{xL}, k_{yL} and k_{VML} is used to identify the uniform loading on a rectangular plate.

(2) The load case is illustrated in Figure B.3.

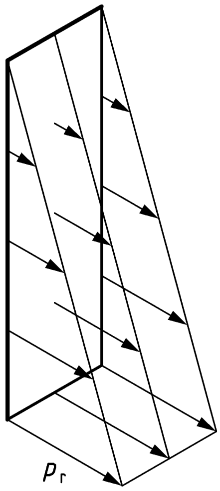


Figure B.3 — Linearly varying pressure on a rectangular plate with reference pressure p_r

B.7.4 Stresses for SCA: FFFF

(1) The moment stress coefficients for SCA are given in Table B.5.

Table B.5 — Moment stress coefficients for boundaries SCA

Moment location	**Moment stress coefficient**
von Mises stress on symmetry axis	$k_{VML} = 0{,}0398\psi^2 - 0{,}1946\psi + 0{,}2584$
von Mises stress on side	$k_{VML} = 0{,}0602\psi^2 - 0{,}381\psi + 0{,}4885$
von Mises stress on base	$k_{VML} = -0{,}0411\psi^2 - 0{,}0938\psi + 0{,}3444$
max sagging x moment on symmetry axis	$k_{xL} = 0{,}027\psi^2 - 0{,}1912\psi + 0{,}2347$
max sagging y moment on symmetry axis	$k_{yL} = 0{,}0036\psi^2 - 0{,}0324\psi + 0{,}1049$
most negative hogging y moment on base	$k_{yL} = 0{,}0395\psi^2 + 0{,}0896\psi - 0{,}3297$
most negative hogging x moment on side	$k_{xL} = -0{,}0576\psi^2 + 0{,}3649\psi - 0{,}4678$

B.7.5 Stresses for SCB: SSSS

(1) The moment stress coefficients for SCB are given in Table B.6.

Table B.6 — Moment stress coefficients for boundaries SCB

Moment location	**Moment stress coefficient**
von Mises stress on symmetry axis	$k_{vmL} = 0{,}2735\psi^2 - 0{,}7681\psi + 0{,}7152$
von Mises stress on side	$k_{vmL} = -0{,}0475\psi^2 + 0{,}0601\psi + 0{,}0123$
von Mises stress on base	$k_{vmL} = 0$
max sagging x moment on symmetry axis	$k_{xL} = 0{,}2868\psi^2 - 0{,}8097\psi + 0{,}6712$
max sagging y moment on symmetry axis	$k_{yL} = -0{,}0969\psi + 0{,}261$
most negative hogging y moment on base	$k_{yL} = 0$
most negative hogging x moment on side	$k_{xL} = 0$

B.7.6 Stresses for SCC: FFFS

(1) The moment stress coefficients for SCC are given in Table B.7.

Table B.7 — Moment stress coefficients for boundaries SCC

Moment location	**Moment stress coefficient**
von Mises stress on symmetry axis	$k_{vmL} = 0{,}0578\psi^2 - 0{,}2115\psi + 0{,}2621$
von Mises stress on side	$k_{vmL} = 0{,}1033\psi^2 - 0{,}4179\psi + 0{,}496$
von Mises stress on base	$k_{vmL} = -0{,}0088\psi^2 - 0{,}1258\psi + 0{,}3517$
max sagging x moment on symmetry axis	$k_{xL} = 0{,}0516\psi^2 - 0{,}2114\psi + 0{,}2386$
max sagging y moment on symmetry axis	$k_{yL} = 0{,}0085\psi^2 - 0{,}0416\psi + 0{,}1076$
most negative hogging y moment on base	$k_{yL} = 0{,}0086\psi^2 + 0{,}1202\psi - 0{,}3367$
most negative hogging x moment on side	$k_{xL} = -0{,}0989\psi^2 + 0{,}4003\psi - 0{,}475$

B.7.7 Stresses for SCD: SFFS

(1) The moment stress coefficients for SCD are given in Table B.8.

Table B.8 — Moment stress coefficients for boundaries SCD

Moment location	Moment stress coefficient
von Mises stress on symmetry axis	$k_{vmL} = 0.039\psi^2 - 0{,}1629\psi + 0{,}2689$
von Mises stress on side	$k_{vmL} = 0{,}0719\psi^2 - 0{,}3356\psi + 0{,}5006$
von Mises stress on base	$k_{vmL} = 0$
max sagging x moment on symmetry axis	$k_{xL} = 0{,}0361\psi^2 - 0{,}1705\psi + 0{,}2401$
max sagging y moment on symmetry axis	$k_{yL} = 0{,}0022\psi^2 - 0{,}024\psi + 0{,}1211$
most negative hogging y moment on base	$k_{yL} = 0$
most negative hogging x moment on side	$k_{xL} = -0{,}0688\psi^2 + 0{,}3214\psi - 0{,}4794$

Annex C
(informative)

Formulae for the plastic reference resistances of unstiffened individual plates and plate assemblies

C.1 Use of this Annex

This Informative Annex provides design formulae for the plastic reference resistances of unstiffened individual plates and plate assemblies.

NOTE National choice on the application of this Informative Annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

C.2 Scope and field of application

(1) This Informative Annex defines the plastic collapse resistances of unstiffened individual plates and plate assemblies that may be used in the design of plates under these two conditions.

(2) The plastic collapse resistances are defined in purely algebraic terms: the relevant loading definitions are given in EN 1991, the relevant relationships between characteristic and design values of resistances should be taken from the main text of this standard.

(3) The scope of this Annex is limited to plate geometries and boundary conditions that are deemed most relevant to the design of unstiffened plates and unstiffened plate or panels of plate assemblies.

NOTE The formulae in this annex provide mechanics formulae only. Their use as characteristic or design values is a matter for the body of the standard and for the designer.

C.3 General

C.3.1 Geometries of individual plates

(1) The geometry of each individual plate may take the following forms as defined in Figure 3.2:

— rectangular;

— trapezoidal;

— triangular.

(2) Where the shape is rectangular, the longer side length is defined as dimension a and the shorter side length as dimension b (Figure 3.2).

(3) Where the shape is triangular, the dimension of the side parallel to the axis of symmetry is defined by the dimension a as shown in Figure 3.2. The length of the side perpendicular to the axis of symmetry is defined by the dimension b.

(4) Where the shape is trapezoidal, the dimensions of the sides parallel to the axis of symmetry are defined as a. The lengths of the sides perpendicular to the axis of symmetry are defined by the dimensions b_1 and b_2 as shown in Figure 3.2.

C.3.2 Load conditions covered in this Annex

(1) The load cases for individual plates treated in this Annex are as follows:

— uniform pressure;

— linear variation from zero to the reference pressure p_r;

— Janssen variation for silo pressures as defined in EN 1991-4.

C.3.3 Boundary conditions for individual plates

(1) The different defined boundary conditions according to Table 6.4 are used, but with two equivalences for additional ease of use with the notation F and S in the sequence: base, two sides, top:

— BC1r or BC2r rotationally restrained edges are both termed F

— BC1f or BC2f rotationally free edges are both termed S

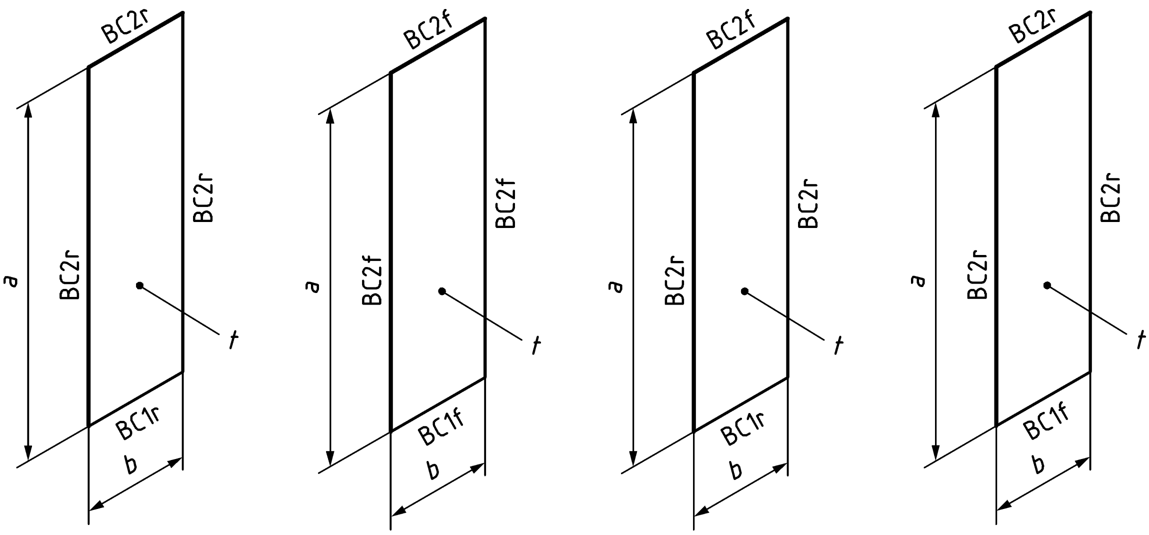
(2) For rectangular plates, the following support conditions (SC) are defined for four boundary condition combinations (see Figure C.1 and Table C.1):

— SCA: All sides rotationally restrained: FFFF

— SCB: All sides rotationally free: SSSS

— SCC: Base and sides rotationally restrained, top rotationally free: FFFS

— SCE: Base rotationally free, sides and top rotationally restrained: SFFF



a) Support condition: SCA FFFF | b) Support condition: SCB SSSS | c) Support condition: SCC FFFS | d) Support condition: SCE SFFF

Figure C.1 — Support conditions for rectangular plates

Table C.1 — Definitions of plate edge boundary conditions

Degree of freedom	**Rotation**			**Translation normal to the plate**	**Translation in the plane of the plate**		
Support condition name	Base	Sides	Top	All edges	Base	Sides	Top
SCA	Clamped	Clamped	Clamped	Restrained	Restrained	Free	Free
SCB	Free	Free	Free	Restrained	Restrained	Free	Free
SCC	Clamped	Clamped	Free	Restrained	Restrained	Free	Free
SCE	Free	Clamped	Clamped	Restrained	Restrained	Free	Free

NOTE Translation of the edge normal to the plane of the plate is restrained on all edges in all conditions.

C.4 Rectangular plates under uniform pressure

C.4.1 Geometry and loading

(1) A rectangular plate under uniform reference pressure p_{r} is shown in Figure C.2.

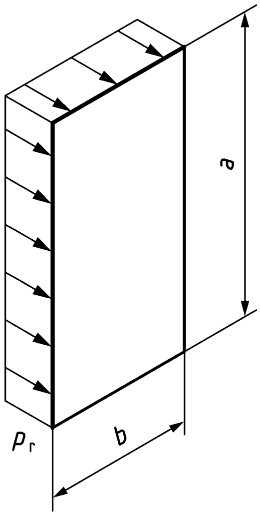


Figure C.2 — Uniform pressure on a rectangular plate with reference pressure p_r

C.4.2 Plastic reference resistances under uniform pressure

(1) The plastic reference resistance $p_{r,pl}$ under uniform transverse pressure is given by:

$$p_{\mathrm{r,pl}} = \beta_U \left(\frac{t^2 f_{\mathrm{y}}}{ab} \right) \tag{C.1}$$

in which:

$$\beta_{\mathrm{U}} = 7{,}38\zeta_1 - 1{,}61\zeta_2 \tag{C.2}$$

where

β_{U} is the plastic reference resistance factor for a plate under uniform pressure.

(2) The values of ζ_1, and ζ_2 are given in Table C.2 and depend on the support conditions (Figure C.1 and Table C.1) and the aspect ratio ψ of the plate.

Table C.2 — Parameters ζ_1 and ζ_2 for all load cases

Case	Support condition	ζ_1	ζ_2
1	SCA	$\zeta_1 = 0{,}53 + 0{,}63\psi + \dfrac{0{,}62}{\psi}$	$\zeta_2 = 1{,}29 + 0{,}21\psi + \dfrac{0{,}16}{\psi}$
2	SCB	$\zeta_1 = 0{,}33 + 0{,}33\psi + \dfrac{0{,}33}{\psi}$	$\zeta_2 = 0{,}7 + 0{,}12\psi + \dfrac{0{,}18}{\psi}$
3	SCC	$\zeta_1 = 0{,}96 + 0{,}15\psi + \dfrac{0{,}49}{\psi}$	$\zeta_2 = 1{,}22 + 0{,}24\psi + \dfrac{0{,}23}{\psi}$
4	SCE	$\zeta_1 = 0{,}97 + 0{,}06\psi + \dfrac{0{,}48}{\psi}$	$\zeta_2 = 0{,}72 + 0{,}35\psi + \dfrac{0{,}34}{\psi}$

C.5 Rectangular plates under linear variation of pressure

C.5.1 General

(1) A rectangular plate under linear pressure variation, characterised by the maximum pressure p_{r}, is shown in Figure C.3.

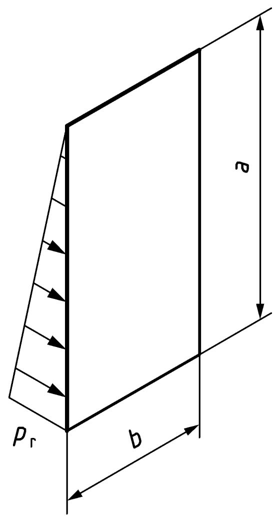


Figure C.3 — Linear pressure variation on a rectangular plate with reference pressure p_r

C.5.2 Plastic reference resistances under linear pressure variation

(1) The plastic reference resistance $p_{r,pl}$ under a linear transverse pressure variation is given by

$$p_{\mathrm{r,pl}} = \beta_L \left(\frac{t^2 f_{\mathrm{y}}}{ab} \right) \tag{C.3}$$

in which

$$\beta_{\mathrm{L}} = 7{,}38\zeta_1 + 3\zeta_2 \tag{C.4}$$

where

β_{L} is the plastic reference resistance factor for a plate under linearly varying pressure.

(2) The values of ζ_1, and ζ_2 are given in Table C.2 and depend on the support conditions (Figure C.1 and Table C.1) and the aspect ratio ψ of the plate.

C.6 Rectangular plates under Janssen pressure variation

C.6.1 General

(1) A rectangular plate under a Janssen pressure variation, as defined in EN 1991-4, is characterised here by the maximum pressure p_r at the base of the plate, as shown in Figure C.4. The Janssen asymptotic pressure p_O is not used in this description.

(2) The Janssen pressure distribution on the plate depends on the rate of change of pressure in this distribution. This rate of change is characterised here using the Janssen reference depth z_O according to EN 1991-4.

NOTE The resulting pressure distribution can be almost linear where z_O is large, or close to constant where z_O is very small.

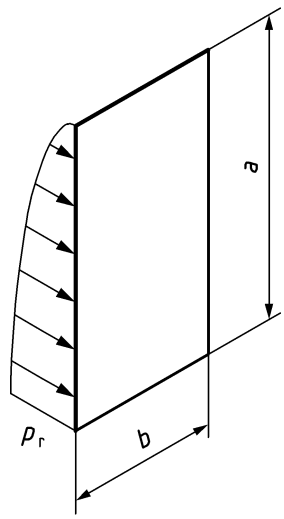


Figure C.4 — Janssen pressure variation on a rectangular plate with reference pressure p_r

C.6.2 Plastic reference resistances under Janssen pressure variation

(1) The plastic reference resistance $p_{r,pl}$ under a Janssen transverse pressure variation is given by:

$$p_{\mathrm{r,pl}} = \beta_J \left(\frac{t^2 f_{\mathrm{y}}}{ab} \right) \tag{C.5}$$

in which

$$\beta_J = 7{,}38\zeta_1 + 3\zeta_2 \tanh\left(\frac{1404}{\psi} \frac{z_{\mathrm{o}}}{a} - 0{,}6 \right) \tag{C.6}$$

where

- β_J is the plastic reference resistance factor for a plate under a Janssen pressure variation;
- z_O is the Janssen characteristic depth (see EN 1991-4).

(2) The values of ζ_1, and ζ_2 are given in Table C.2 and depend on the support conditions (Figure C.1 and Table C.1) and the aspect ratio ψ of the plate.

Bibliography

Other references

The following documents are those not included in the above categories but are cited informatively in the document, for example in notes.

[1] ECCS EDR5 (2013) European Recommendations for Steel Construction: Buckling of Shells, 5th edition revised second impression, Edited by J.M. Rotter and H. Schmidt, European Convention for Constructional Steelwork, Brussels, 388 pp.

[2] EN 1998 (all parts), *Eurocode 8: Design of structures for earthquake resistance*