CEN/TC 250/SC 3 N 2640

CEN/TC 250/SC 3
Eurocode 3 - Design of steel structures
E-mail of Secretary: susan.kempa@din.de
Secretariat: DIN

EN 1993-1-7 First Draft

Date of document	2018-05-02
Expected action	Comment
Due Date	2018-06-29

Background

Dear Member,

Please find attached the First Draft of the revision of EN 1993-1-7 for commenting. Please ensure that you write any comments into the CEN comments template and forward them to your NSB for submission via the CIB ballot. The CIB ballot will close on 29th June 2018. We kindly ask for giving comments only to the obviously changed parts and not to the original document text.

Kind regards

Susan Kempa
Secretary CEN/TC 250/SC 3

EUROPEAN STANDARD

NORME EUROPÉENNE

EUROPÄISCHE NORM

prEN1993-1-7: 2018

April 2018

UDC

Descriptors:

English version

Eurocode 3: Design of steel structures – Part 1-7

Plate assemblies with elements under transverse loads

Calcul des structures en acier	Bemessung und Konstruktion von Stahlbauten
Partie 1.7 :	Teil 1.7 :
...	Ergänzende Regeln zu ebenen Blechfeldern mit Querbelastung

Not yet transferred to the new template

Version V13

April 2018

CEN

European Committee for Standardisation

Comité Européen de Normalisation

Europäisches Komitee für Normung

Central Secretariat: rue de Stassart 36, B-1050 Brussels

© 2018 Copyright reserved to all CEN members

Ref. No. EN 1993-1.7 : 2013. E

Contents

Foreword

This European Standard EN 1993-1-7, Eurocode 3: Design of steel structures: Part 1-7 Plate assemblies with elements under transverse loads, has been prepared by Technical Committee CEN/TC250 « Structural Eurocodes », the Secretariat of which is held by BSI. CEN/TC250 is responsible for all Structural Eurocodes.

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publication of an identical text or by endorsement, at the latest by xxxxx, and conflicting National Standards shall be withdrawn at latest by **** .

This Eurocode supersedes EN 1993-1-7 (2007).

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

National annex for EN 1993-1-7 (20**)

There are no Nationally Determined Parameters associated with EN 1993-1-7.

This standard gives alternative procedures, values and recommendations with notes indicating where national choices may have to be made. The National Standard implementing EN 1993-1-7 should have a National Annex containing all Nationally Determined Parameters to be used for the design of steel structures to be constructed in the relevant country.

National choice is allowed in EN 1993-1-7 through:

EDITORIAL NOTE: IT IS EXPECTED THAT THERE WILL BE NO NDPs IN THIS STANDARD

1 Scope

(1)P EN 1993-1-7 provides basic design rules for the structural design of assemblies of unstiffened and stiffened plates whose plate elements or segments are under predominantly transverse loads. These include containment structures such as silos, tanks, digesters and lock gates, where the external actions chiefly act transversely on individual plates, but whose plate elements or segments also carry membrane forces in their plane due to the connectivity between the component plates. It is intended to be used in conjunction with EN 1993-1-1, EN 1993-1-8, EN 1993-1-5, EN 1993-4-1, EN 1993-4-2, and other relevant application standards.

(2) This document defines the design values of the resistances; the partial factor for resistances may be taken from National Annexes of the relevant application standards. Recommended values are given in the relevant application standards.

(3) This Standard is concerned with the requirements for design against the ultimate limit states of:

– plastic failure;

– cyclic plasticity;

– buckling;

– fatigue.

(4) Overall equilibrium of the structure (sliding, uplifting, or overturning) is not included in this Standard, but is treated in EN 1993-1-1. Special considerations for specific applications may be found in the relevant applications parts of EN 1993.

(5) The rules in this Standard refer to plate assemblies that may be fabricated using unstiffened or stiffened plate segments. The standard may also be used to design individual plate segments. The individual plate segments are principally subject to actions transverse to the plane of each plate, but both frictional actions on the plate surface and forces imposed by adjacent components of the plate assembly may also induce in-plane actions in each plate.

(6) For the verification of unstiffened and stiffened plate assemblies subject only to in-plane forces, see EN 1993-1-5. This standard, EN 1993-1-7, gives rules/guidance to account for the interaction between bending and membrane forces in the individual plate segments.

(7) For the design rules for cold formed members and sheeting, see EN 1993-1-3.

(8) The temperature range within which the rules of this Standard may be applied are defined in the relevant application parts of EN 1993. Where there is no relevant application part, the temperature range should be taken to be design metal temperatures within the range –50°C to +150°C.

(9) The rules in this Standard refer to structures constructed in compliance with the execution specification of EN 1090-2.

(10) Where the plate assembly is subject to time-varying actions such as wind loading and bulk solids flow, these should be treated as quasi-static actions. For high cycle varying actions, the susceptibility of the structure to fatigue should be taken into account according to any relevant clauses in this code and EN 1993-1-9. The stress resultants arising from all dynamic behaviour are treated in this part as quasi-static.

2 Normative references

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

EN 1993 Eurocode 3:Design of steel structures:

Part 1.1: General rules and rules for buildings

Part 1.3: Cold-formed members and sheeting

Part 1.4: Stainless steels

Part 1.5: Plated structural elements

Part 1.6: Strength and stability of shell structures

Part 1.8: Design of joints

Part 1.9: Fatigue strength of steel structures

Part 1.10: Selection of steel for fracture toughness and through-thickness properties

Part 1.12: Additional rules for the extension of EN 1993 up to steel grades Sxxx

Part 4.1: Silos

Part 4.2: Tanks

3 Terms and definitions

3.1 General

(1) The rules in EN 1990, clause 1.5 apply.

(2) The following terms and definitions are supplementary to those used in EN 1993-1-1.

3.2 Structural forms and geometry

3.2.1

plate assembly

a structure that is assembled from flat plates which are joined together (see Figure 3.1). The shape of the individual plates may be rectangular, triangular or trapezoidal. It is assumed that the assembly has at least one axis of symmetry . The individual plates may be unstiffened or stiffened (see Figure 3.2).

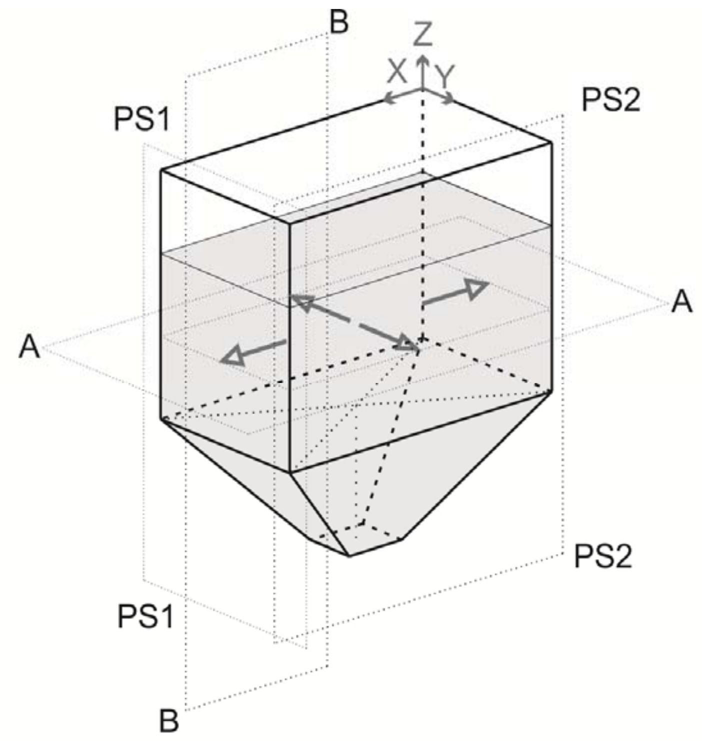


Figure 3.1 Typical arrangement of an assembly of flat, unstiffened or stiffened plates with global coordinate system and characteristic sections

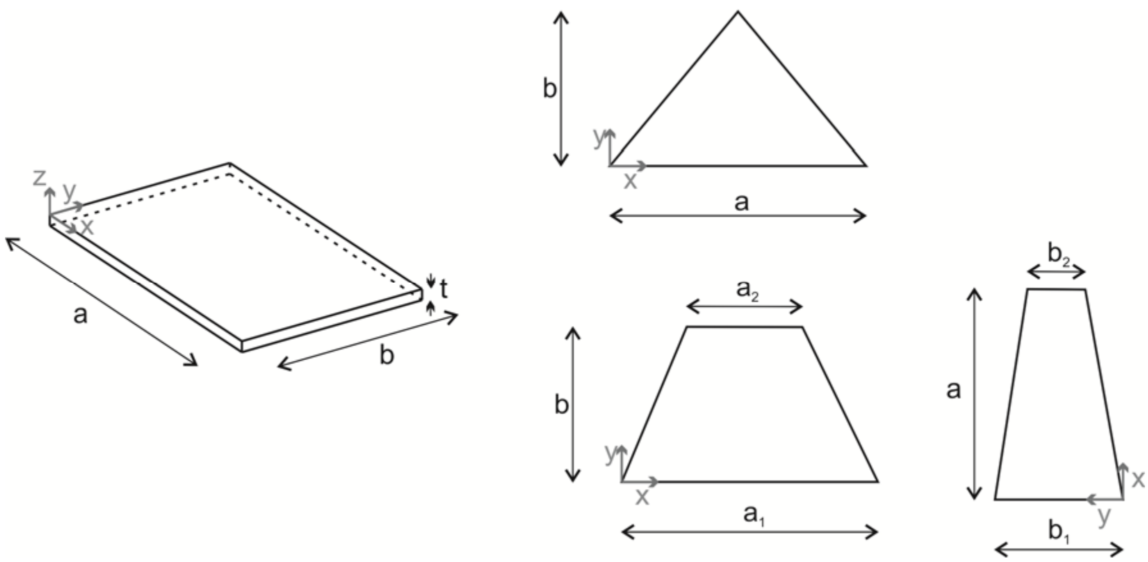


Figure 3.2 Dimensions and local coordinate systems for rectangular, triangular and trapezoidal plates

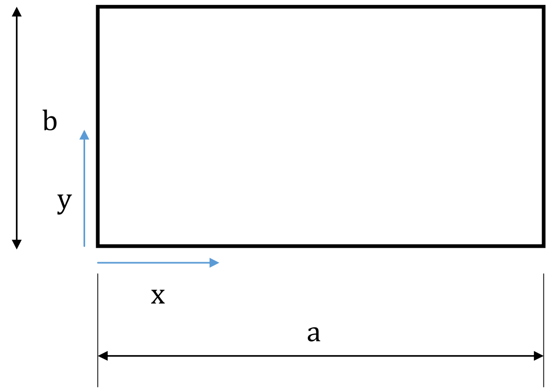


Figure 3.3 Dimensions and coordinate system for the reference rectangular plate

3.2.2

plate segment

a flat plate which may be unstiffened or stiffened. A plate segment is a single component of a plate assembly. A plate segment should be regarded as an individual part of a plate assembly (see Figure 3.2)

3.2.3

stiffener

a flat plate or prismatic member attached to a plate segment for the purpose of increasing its bending resistance. It may also be used to reinforce it to support local loads.

3.2.4

longitudinal stiffener

a stiffener on a rectangular plate segment in which the stiffener longitudinal axis is aligned with the longer dimension, *a* of the plate segment (see Figure 3.4)

3.2.4

transverse stiffener

a stiffener on a rectangular plate segment in which the stiffener longitudinal axis is aligned with the shorter dimension, *b* of the plate segment (see Figure 3.4)

3.2.5

uni-directionally stiffened plate

a plate that has stiffeners attached to it with their longitudinal axis in a single direction. The direction can be longitudinal or transverse

3.2.6

bi-directionally stiffened plate

a plate that has stiffeners attached to it with their longitudinal axes in a two orthogonal directions

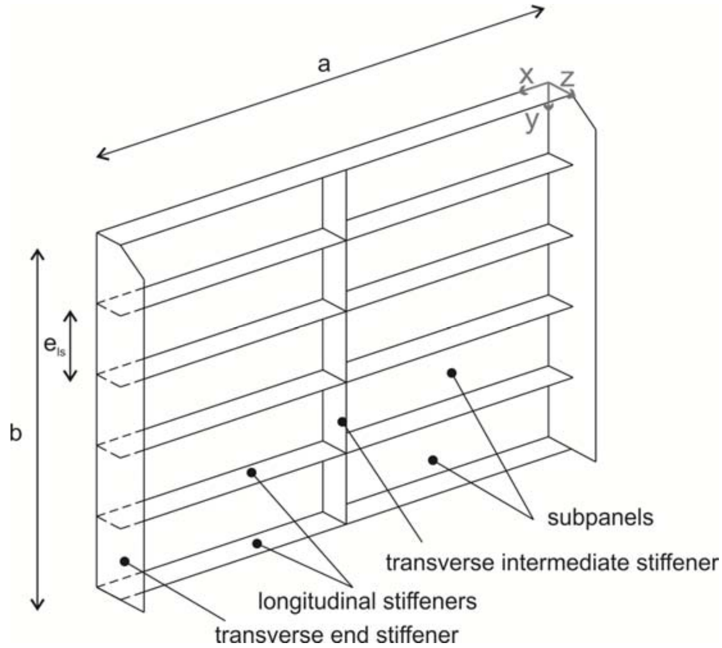


Figure 3.4 Example of a rectangular stiffened plate

3.2.5

subpanel

the part of a stiffened plate that lies between stiffeners, and so is locally an unstiffened plate bounded by stiffeners

3.3 Failure mechanisms

3.3.1

plastic failure

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

EDITORIAL COMMENT: WE NEED THE TERMINOLOGY TO BE CONSISTENT WITH EN 1993-1-6 WHERE WE REMOVED THE TERMS "PLASTIC LIMIT" AND "PLASTIC COLLAPSE" BECAUSE THIS IS NOT THE LIMIT LOAD OF IDEAL PLASTICITY, NOR IS IT A COLLAPSE LOAD WITH A STABLE PLATEAU (as in a beam). MANY INSTANCES OCCUR WHERE THE STRUCTURE CONTINUES TO DEFORM AND BEAR LARGER LOADS UNTIL THE DEFORMATIONS ARE DEEMED UNACCEPTABLE. THIS HAPPENS IN SHELLS, BUT CAN BE EVEN MORE THE CASE IN UNSTIFFENED PLATE ASSEMBLIES.

3.3.2

tensile rupture

the ultimate limit state where separation of the parts of a plate segment or the junctions between plate segments occurs due to tension

3.3.3

cyclic plasticity

the ultimate limit state in which repeated cycles of loading lead to repeated plastic straining. Two distinct failure modes may arise: ratcheting and low-cycle fatigue

3.3.4

ratcheting

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

3.3.5

low cycle fatigue

the ultimate limit state where repeated alternating cycles of plastic strain cause fatigue cracking

3.3.6

buckling

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

3.3.7

high cycle fatigue

the ultimate limit state where a high number of cycles of loading and unloading under elastic stresses cause fatigue cracking

3.4 Actions

3.4.1

transverse loading

a load applied normal to the middle surface of a plate segment

3.4.2

in-plane loading

forces applied parallel to the middle surface of a plate segment. They may be applied through the connections between the plate segment and other plate segments, 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 plate segment to be carried by forces in the plane of the plate segment

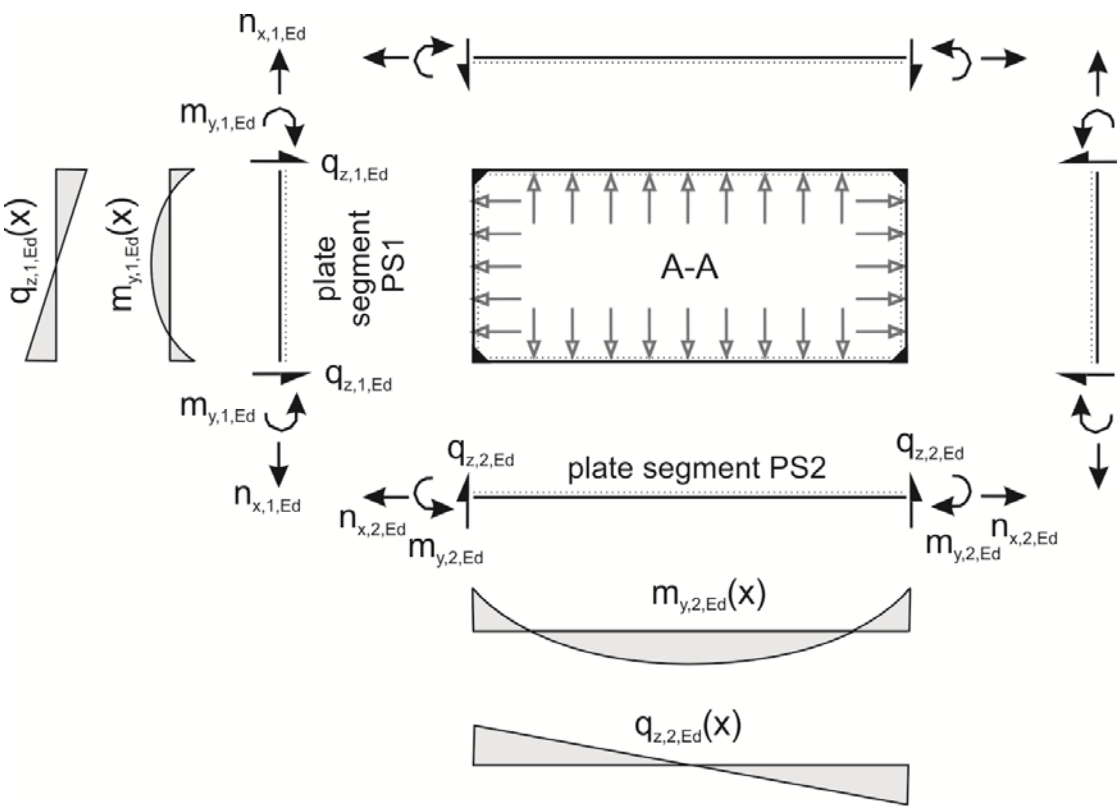


Figure 3.5 Plate internal forces and moments on Section A-A in Figure 3.11

3.5 Symbols

In addition to those given in EN 1990 and EN 1993-1-1, the following symbols are used:

(1) Membrane stress resultants in a plate segment (see Figures 3.5 and 3.6)

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

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

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

(2) Membrane stresses in a plate segment (see Figures 3.5 and 3.6):

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

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

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

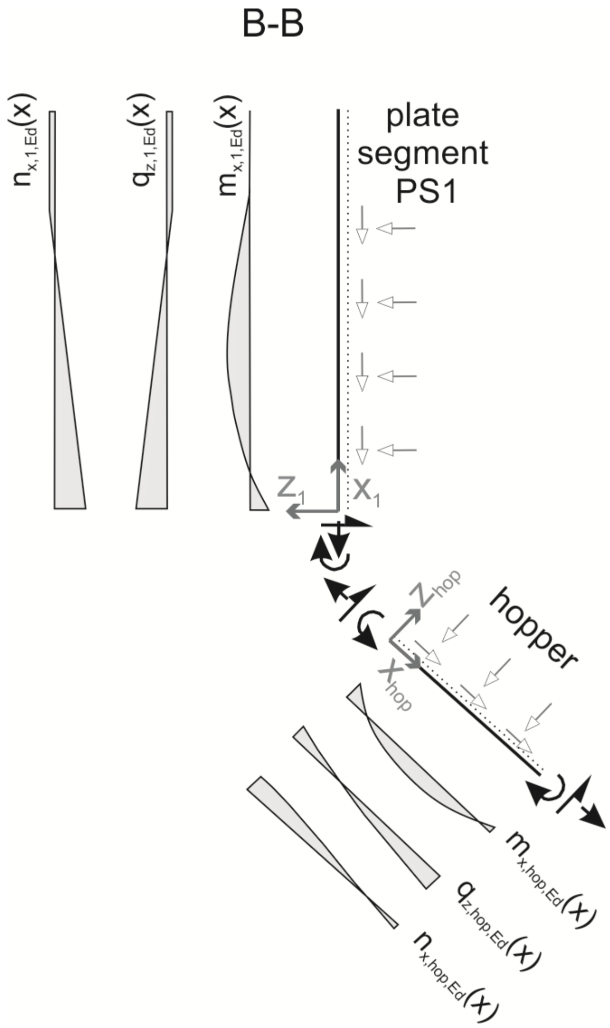


Figure 3.6 Plate internal forces and moments on Section B-B in Figure 3.1

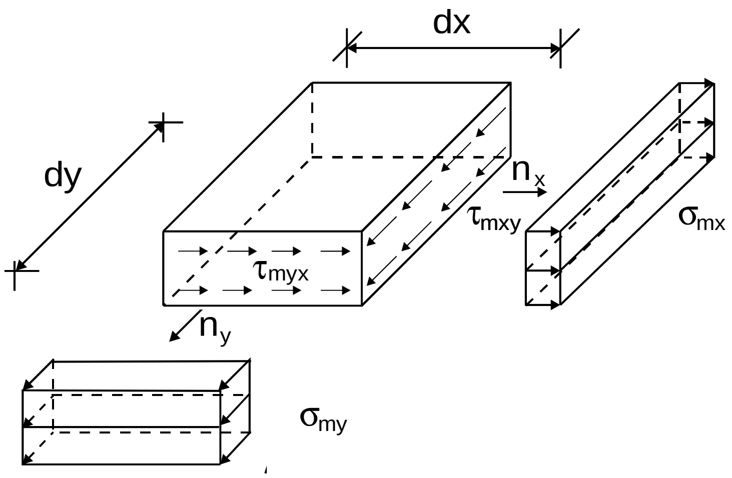


Figure 3.7 Membrane stresses and membrane stress resultants in a plate segment

to be improved

(3) Bending and twisting stress resultants in a plate segment (see Figure 3.8)

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

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

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

(4) Bending and shear stresses in plate segments due to bending (see Figure 3.8)

σ_{bx} is the bending stress in the x-direction due to bending moment per unit width m_x: positive for tensile stresses on the positive z axis face;

σ_{by} is the bending stress in the y-direction due to bending moment per unit width m_y: positive for tensile stresses on the positive z axis face;

τ_{bxy} is the shear stress due to the twisting moment per unit width m_{xy}: positive for positive shear stresses on the positive z axis face;

(5) Transverse shear stresses in plate segments (see Figure 3.8)

τ_{bxz} is the shear stress due to transverse shear forces per unit width q_x associated with bending: positive for shear in the z direction;

τ_{byz} is the shear stress due to transverse shear forces q_y associated with bending: positive for shear in the z direction;

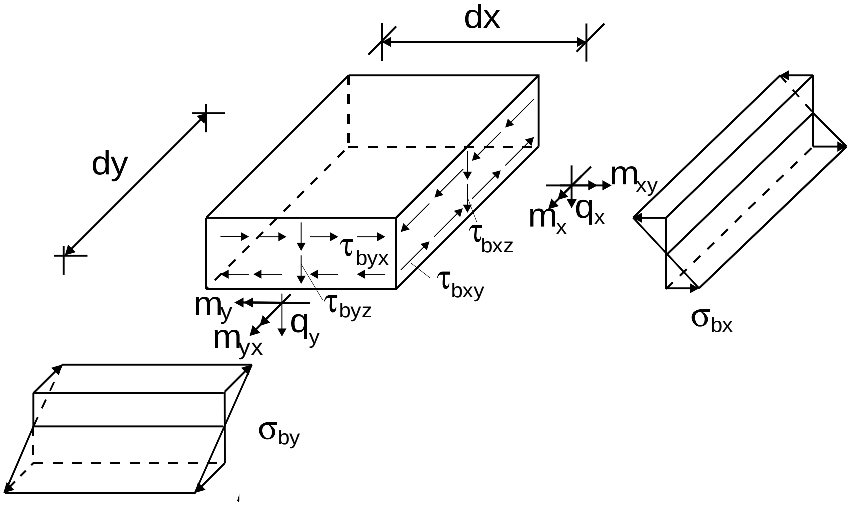


Figure 3.8 Bending stresses and bending moments in a plate segment

to be improved

NOTE: In general, there are eight stress resultants in a plate at any point. The shear stresses τ_{bxz} and τ_{byz} due to q_x and q_y are in most practical cases negligible compared to the other components of stress, and therefore they may normally be disregarded for the design.

(6) Latin upper case letter:

E modulus of elasticity

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

R_{cr} elastic critical resistance of the structure subject the design values of loads in a specific load case

R_k characteristic value of the ultimate resistance of the structure subject the design values of loads in a specific load case

R_{pl} plastic reference resistance of the structure subject to the design values of loads in a specific load case

(7) Latin lower case letters:

a length of a rectangular plate segment (longer dimension), see Figure 3.3

b width of a rectangular plate segment (shorter dimension), see Figure 3.3

a_1 base of a symmetrical triangular or trapezoidal plate segment where $a_1 \geq b$, see Figure 3.2

a_2 width of the top of a symmetrical triangular or trapezoidal plate segment where $a_1 \geq b$, see Figure 3.2

b_1 base of a symmetrical triangular or trapezoidal plate segment where $a_1 < b$, see Figure 3.2

b_2 width of the top of a symmetrical triangular or trapezoidal plate segment where $a_1 < b$, see Figure 3.2

f_{yk} yield stress or 0,2% proof stress for material with non linear stress-strain curve;

t thickness of a plate segment, see Figure 3.2.

Note: The dimension "*a*" is defined in different senses in common texts on plates, making no single notation universal. In this standard the use of "*a*" as the longer side of any plate element aims to provide consistency, especially with EN 1993-1-1.

(8) Greek lower case letters:

α aspect ratio of a rectangular plate segment (*b/a*) (*≤1,0*);

ε strain;

ρ reduction factor for plate buckling;

ν Poisson's ratio;

γ_M partial factor on resistance.

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

4 Basis of design

4.1 General

(1)P The basis of design shall be in accordance with EN 1990 and EN 1993-1-1, as supplemented by the following.

(2)P 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 serviceability requirements set out in 11 or relevant application and product standards.

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

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

4.2 Reliability management

(1) The execution classes of EN 1090-2 and EN 1090-4 for a plate assembly should be selected in accordance with Annex A of 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 EN1990. 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 and EN 1993-4-2).

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, reduced by the prescribed value of any corrosion or abrasion loss, should be used.

(2) The thickness ranges within which the rules of this standard may be applied are defined in the relevant EN 1993 application parts.

(3) The middle surface of each plate segment should be taken as the reference surface for applied loads unless stated otherwise.

4.4 Geometrical tolerances and geometrical imperfections

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

(2) When the limit state of buckling (LS3, see 7.1.3) is the limit state to be considered, the additional buckling-relevant geometrical tolerances given in Annex “X” should be used.

4.5 Durability

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

5 Materials

5.1 Material properties

(1) This Standard 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 product standards listed therein.

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

(4) In a global numerical analysis using material nonlinearity, the 0,2% proof stress should be used to represent the yield stress f_y in all relevant expressions. The stress-strain curve should be modelled in accordance with EN1993-1-5 Annex C for carbon steels and EN1993-1-4 Annex C for stainless steels.

(5) Where materials with nonlinear stress-strain curves are involved and a buckling analysis is carried out under stress design (see 8.5), the initial tangent value of Young´s modulus E should be replaced by a reduced value. If no better method is available, the secant modulus at the 0,2% proof stress should be used when assessing the quasi-elastic buckling resistance or quasi-elastic buckling stress.

6 Structural analysis

6.1 Types of analysis

6.1.1 General

(1) This code assumes that plate assemblies will generally be designed using a global numerical analysis (for example, by means of computer programs such as those based on the finite element method) - see 6.2.

(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 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	not applicable	perfect
Linear elastic plate assembly analysis (LA)	linear bending and stretching	linear	perfect
Linear elastic bifurcation analysis (LBA)	linear bending and stretching	linear	perfect
Geometrically non-linear elastic analysis (GNA)	non-linear (deformed geometry)	linear	perfect
Materially non-linear analysis (MNA)	linear bending and stretching	ideal elastic plastic	perfect
Geometrically and materially non-linear analysis (GMNA)	non-linear (deformed geometry)	fully non-linear	perfect
Geometrically and materially non-linear analysis with imperfections (GMNIA)	non-linear (deformed geometry)	fully non-linear	imperfect

6.1.2 Membrane and simple bending analysis (MSBA)

(1) A membrane and simple bending analysis treats each plate segment as separate, with a simple one-way beam bending treatment as a strip in each plate segment and only simple end shear forces in one plate being transmitted as membrane forces into the adjacent plate segment (see Figures 3.1 and 3.2).

(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 Notes on Boundary Conditions, see 6.2.3

(3) A membrane and simple bending analysis does not necessarily fulfil the compatibility of deformations at boundaries or within the plate segment or between plate segments either of different shape or that are subjected to different patterns of loading. However, the resulting membrane forces satisfy the equilibrium requirements of primary stresses (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). Small deflection theory implies that the assumed geometry remains that of the un-deformed structure.

(2) An LA analysis satisfies compatibility in the deformations between plate segments as well as equilibrium. The resulting field of membrane and bending stresses satisfies the requirements of primary plus secondary stresses (LS2 and LS4).

6.1.4 Linear elastic bifurcation analysis (LBA)

(1) The conditions of 2.2.5 concerning the material and geometric assumptions are met. However, linear bifurcation analysis obtains the lowest eigenvalue at which the elastic plate assembly may buckle into a different deformation 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. This analysis provides the elastic critical buckling resistance R_{cr} see x.x, x.x and x.x (LS3).

6.1.5 Geometrically nonlinear elastic analysis (GNA)

EDITORIAL NOTE: CJB Not sure where the suggestion that this section should be omitted came from. Discuss?

JMR thinks that this sub-clause should definitely be included, because elastic analysis is the natural default treatment for the designer, and GN makes a huge difference. So this should not be left out.

(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 matches the definition of primary plus secondary stresses (LS2 and LS4, see 7.1.2 and 7.1.4).

(2) Where changes of geometry caused by the loads produce significant redistributions in the elastic stress state, this analysis may be of considerable assistance in 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 buckling load of the perfect structure, including changes in geometry, that may be of assistance in checking the limit state LS3 - see 7.1.3.

(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.

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} used in x.x, x.x and x.x.

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

(3) 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, see 7.1.1 and 7.1.2

(2) Where compression or shear stresses are predominant in some part of the plate assembly, a GMNA analysis gives the elastic-plastic buckling load of the perfect structure that may be of assistance in checking the limit state LS3, see 7.1.3.

(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.

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 loads for the imperfect structure, which may be used for checking the limit state LS3, see 7.1.3.

(2) 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.

(3) 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.

6.2 Modelling of a plate assembly for analysis

6.2.1 General

(1) Most plate assemblies subject to transverse loads will behave in a complex manner. Unless modelling simplifications can be made with confidence

6.2.2 Geometry

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

(2) Imperfections in the plates should be neglected, except when verifying the buckling limit state (LS3, see 7.1.3).

(3) An assembly of plate segments should not be subdivided into separate plates for analysis unless the boundary conditions for each segment are chosen in such a way as to represent interactions between them in a conservative manner.

EDITORIAL NOTE from CJB: This is an uncomfortable statement for me. IT IS CLEAR THAT MANY CASES CAN BE EASILY DEFINED – i.e. THE POSITION OF MAXIMUM MOMENTS CAN BE PREDICTED WITH CONFIDENCE. in other cases THE POSITION OF MAXIMUM MOMENT IS LESS CLEAR – e.g. WHERE IS THE MAXIMUM MOMENT IN A SILO WALL PLATE or in a TANK WALL (i.e. JANSSEN OR HYDROSTATIC LOADING) THAT IS NEARLY SQUARE AND WITH A FREE UPPER EDGE?

(4) If the boundary conditions can be conservatively defined, i.e. restrained or unrestrained, a plate assembly may be subdivided into individual plate segments that may be analysed independently. The interactions between the plate segments should be examined to determine the actions that each applies to its neighbours (see Figure 3.2, Figure 3.4)

(5) Base beams may be used to transfer local support forces into a plate assembly. These should not be separated from the plate assembly that it supports in an (the, any?) assessment of limit state LS3 (see 7.1.3).

Note 1 The base beam flexural stiffness is often lower than the membrane stiffness of the plate above it in compatible deformation. A very stiff base beam is therefore usually required to achieve any effective supporting role.

Note 2 The torsional stiffness of the supporting beam may play an important role, even when it is located symmetrically beneath the plate that it supports.

Note 3 In some applications large in-plane shear forces may develop near supports.

(6) Eccentricities and steps in the middle surface of a plate should be included in the analysis model if they induce significant bending effects as a result of the membrane stress resultants following an eccentric path.

EDITORIAL NOTE FROM CJB: THIS IS ALSO AN UNCOMFORTABLE STATEMENT FOR ME. GREAT CARE IS NEEDED IN MODELLING ANY STEPS IN A PLATE WALL. HOW ARE THESE STEPS TO BE MODELLED.

(7) In the analysis of the plate assembly, a plate segment that has discrete stiffeners attached to it may be treated as an orthotropic uniform plate, or it may be treated as a grillage. (see Note).

EDITORIAL NOTE:_This code will not say any more about orthotropic plates and their modelling so is it appropriate to just say you can do it with no further guidance. The same with the grillage. I would suggest we put a note in here to say it can be done but you need to know what you are doing.

(8) In the analysis of the plate assembly, a plate segment that is corrugated (vertically or horizontally) may be treated as an orthotropic uniform plate

EDITORIAL NOTE:_This code will not say any more about orthotropic plates and their modelling; no simple rules will be provided. These walls are often used in French silos but there are no readily available reliable rules as far as I am aware. This really needs input from practitioners if any guidance is to be provided.

(9) A hole in a plate may be neglected in the modelling, provided its largest dimension is smaller than 5t.

EDITORIAL NOTE:_This is an estimate, and guidance would be welcome.

(10) The overall stability of the complete plate assembly structure should be verified as detailed in the relevant application standards of EN1993.

6.2.3 Boundary conditions

(1) The boundary conditions assumed in the design calculation should be chosen in such a way as to ensure that they achieve a realistic or conservative 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.

(2) In plate buckling (eigenvalue) calculations (limit state LS3, see 7.1.3), the definition of the boundary conditions should refer to the incremental displacements during the buckling process, and not to total displacements induced by the applied actions before buckling.

(3) 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 assembly middle surface. Reference should be made to EN1993-4-1 for silos and to EN 1993-4-2 for the detailed application of this rule to tanks.

(4) Rotational restraints at plate segment boundaries should be included in modelling for limit states LS2 and LS4, see 7.1, but may be neglected in modelling for limit state LS1.

EDITORIAL NOTE – SEE NOTE ATTACHED TO 6.2.2 (3). MUCH FURTHER DISCUSSION ABOUT SAFE TREATMENT OF PLATE ASSEMBLY REQUIRED

(5) In global numerical analyses and in selecting expressions from Annexes A to D, the appropriate boundary conditions should be used in analyses for the assessment of limit states according to the conditions shown in Table 6.1. For the special conditions needed for buckling calculations, reference should be made to 8.4, 9.4 or 10.4 as appropriate.

6.2.4 Modelling of plate junctions

(1) When a global numerical 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.

NOTE Adjacent plates are connected using a range of different details, which may be flexible or stiff and lead to significantly different results in the transfer of forces and moments between plate segments.

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

(3) Unless special provision is made, the junctions should be designed to transmit the full forces and moments associated with rigid joints.

NOTE Unless the forces and moments associated with fully rigid joints are used, the possibility of joint failures due to plastic failure (LS1) and cyclic plasticity (LS2) cannot be eliminated.

(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.

(5) If a stiffened plate assembly is subdivided into individual plate segments the boundary conditions assumed for stiffeners in individual plate segments in the design calculations should be recorded in the drawings and project specification.

EDITORIAL COMMENT: Why only the boundary conditions and not actions and other items

6.3 Modelling of actions and environmental influences

(1) Actions should all 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) Local actions and local patches of action should not be represented by equivalent uniform loads.

(3) The modelling should account for whichever of the following are 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;
- wind effects on openings and penetrations;
- interaction of wind effects on groups of structures;
- 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 design models defined here. These models give a conservative treatment of the plate assembly.

(2) The typical generic forms of plate assemblies considered by this standard are illustrated in Figure 6.1. These may be categorised as assemblies of plates of rectangular, trapezoidal or triangular shape, each plate element having at least one axis of symmetry.

(3) The interactions between plate segments of shapes other than rectangular are of more complex forms, and should be treated using appropriate global numerical analysis.

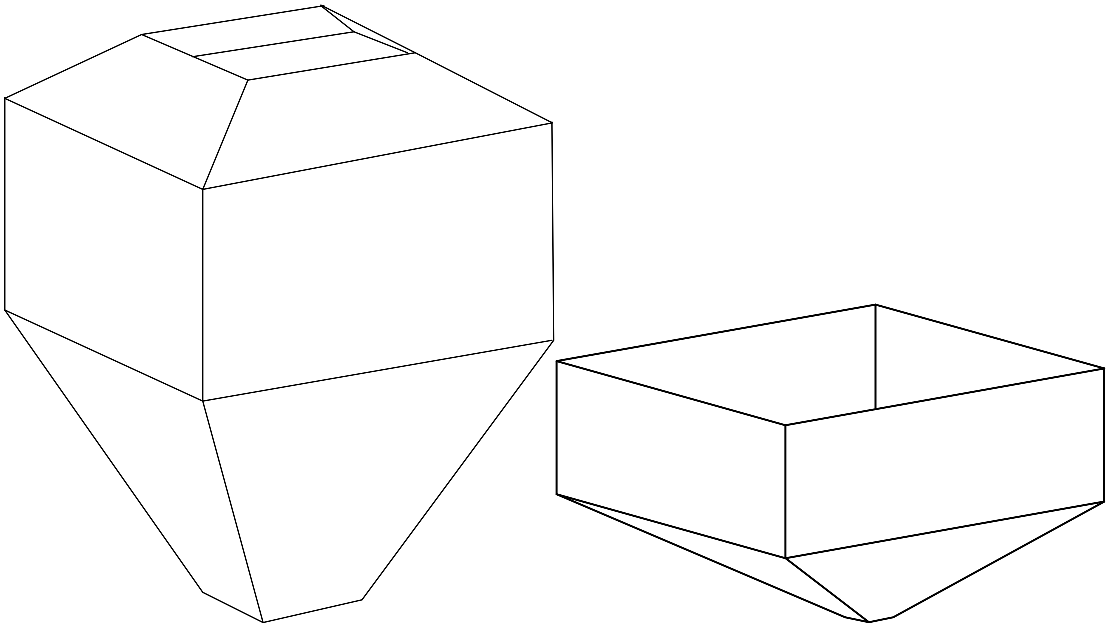


Figure 6.1 Typical arrangement of an assembly of flat, unstiffened or rectangular assemblies

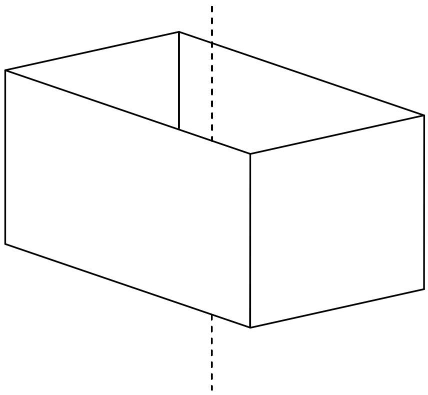


Figure 6.2: Typical rectangular plate assembly

(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 strip of plate subject to relatively constant pressure may be deemed to act so that the end shear on the strip is transmitted to the adjacent plate as a tensile membrane force (see Figure 6.2).

(5) The interactions between plate segments in bending may be treated using a simple analysis that assumes that each plate has a length and mean stiffness that permits analysis as a simple rectangular frame (see Figure 6.4).

(6) to be continued .

EDITORIAL NOTE:- MORE MAY OR MAY NOT BE ADDED HERE DEPENDING ON THE DISCUSSION ON CONSERVATIVE TREATMENTS

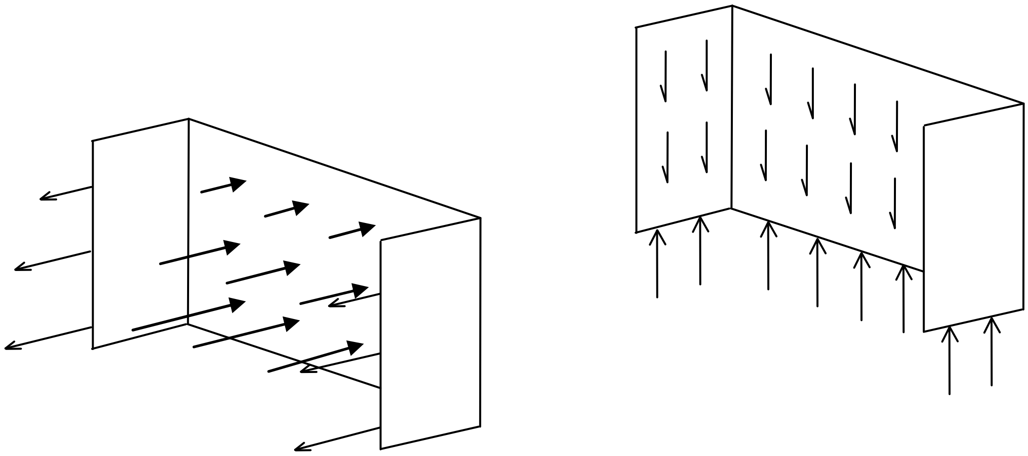


Figure 6.3 Membrane force transmission in a rectangular assembly

Further figures to be drawn

Figure 6.4 Bending moment transmission in a rectangular plate assembly

(7) Rectangular plate assemblies are commonly supported at discrete points, frequently at the vertical boundaries between plate segments (see Figure 6.5).

(8) Where a single plate forms the side of an assembly and the assembly is discretely supported at the two ends of this plate (see Figure 6.5 and 6.2.1), this plate may be required to function as a deep beam spanning between supports, or a structural beam may be introduced beneath the relevant plate. Where a structural beam is used, its stiffness relative to the stiffness of the plate above should be carefully evaluated to guarantee that it carries the intended forces. These two structural arrangements should be treated differently as defined in the following.

NOTE The stiffness of the plate in-plane may be very high compared to the stiffness of any supporting structural elements, and care should be taken to ensure that the loads are transmitted as expected.

(9) The stiffness of the plate above the supports depends on its geometry and the boundary condition at its upper edge.

(10) To be continued with equations to define the stiffnesses and the effectiveness of a base beam.

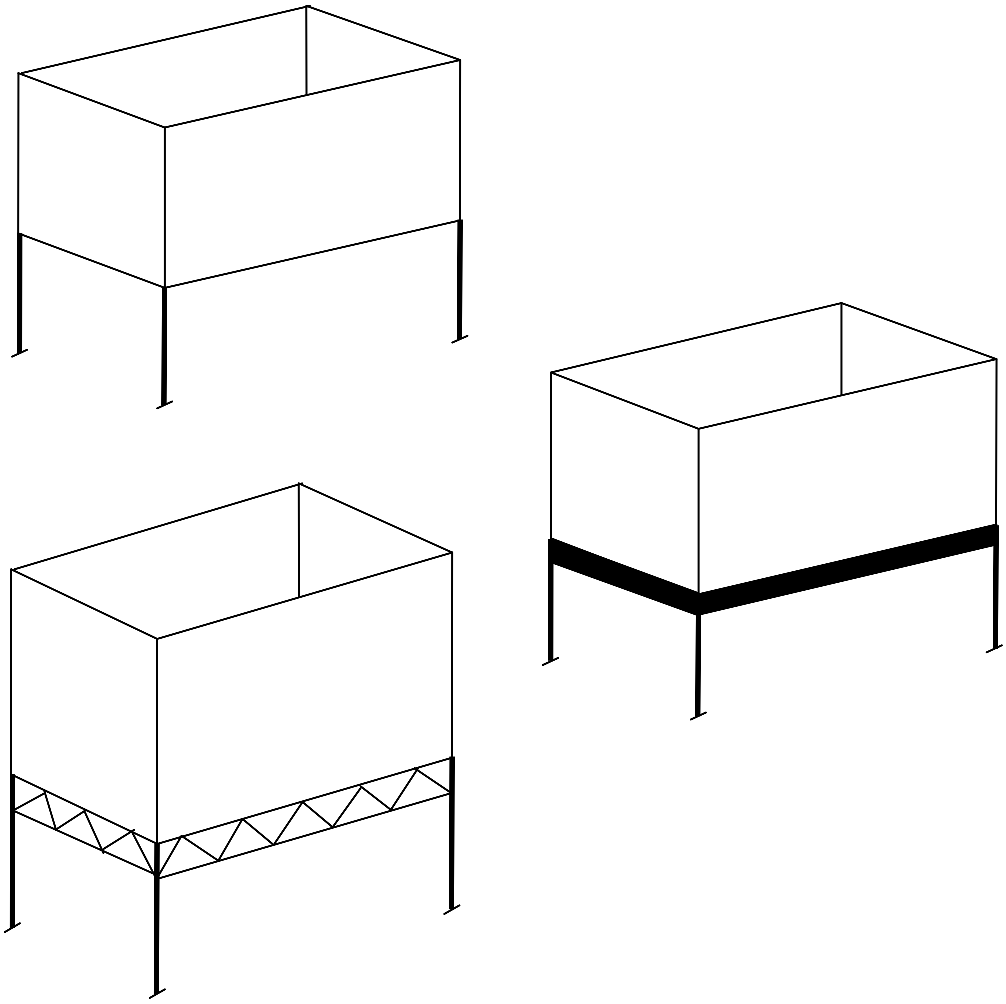


Figure 6.5 Support conditions for rectangular assemblies

6.4.2 Pyramidal assemblies

(1) The transverse loads on the trapezoidal and triangular plates of a pyramidal assembly may be treated as .... text to be completed with some equations ...

(2) The end shear on the strip is transmitted to the adjacent plate as a tensile membrane force (see Figure 6.2).

NOTE: This arrangement of plates produces a very stiff and strong structure, leading to significantly reduced forces and moments in the plate segments, that should be evaluated using the provisions presented here.

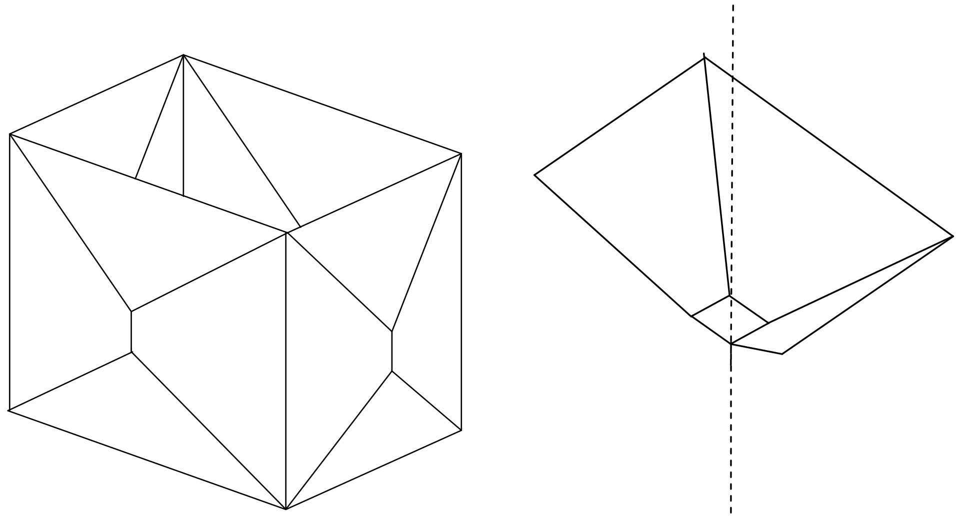


Figure 6.6: Pyramidal assemblies

The above figures to be modified and increased in variety and relocated where appropriate with triangular plates

6.4.3 Analysis of individual plate segments

(1) The individual plate segments of a plate assembly should be analysed after the actions on them from other plates in the assembly have been evaluated.

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

(3) For a rectangular plate, the long dimension is defined as a, and the short dimension as b, with the local x coordinate direction in the longitudinal direction, and the right hand screw rule used to define the coordinates, displacements, forces and moments in the plate. The moments per unit width that induce stresses in the x direction are defined as m_x.

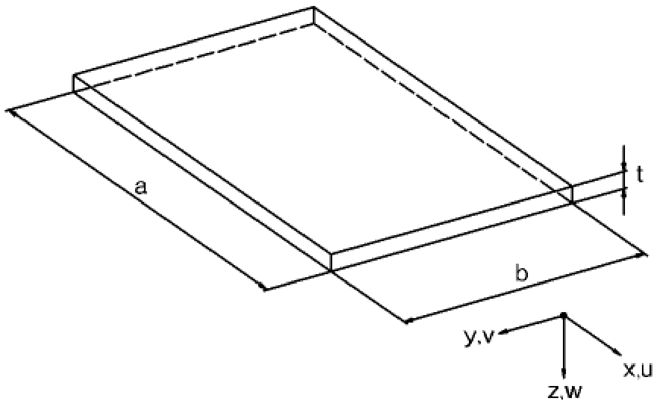


Figure 6.7 Dimensions and coordinate system for a rectangular plate

EDITORIAL NOTE: RE-DRAW fig 3.2 and leave this one?

(3) The simple set of boundary conditions defined for each plate segment are defined in Table 6.1. Nevertheless, the plate segments may interact with each other so that the simple definitions given in Table 6.1 are not relevant. In these cases, the fuller interactions should be considered and a global analysis (e.g. GMNIA) is recommended.

Table 6.1: Boundary conditions for plate edges

Something has gone wrong with the formatting here. The table cannot be made to be associated with its title. This will be fixed for the Second Draft.

Boundary condition code	Simple term	Description	Displacements normal to the plate surface	Displacements in the plane of the plate, normal to the edge	Rotation of the plate edge about the edge
BC1r	Clamped	normal displacements restrained in-plane restrained rotation restrained	*w* = 0	*u* = 0	□□ = 0
BC1f		normal displacements restrained in-plane restrained rotation free	*w* = 0	*u* = 0	□□ □ 0
BC2r		normal displacements restrained in-plane free rotation restrained	*w* = 0	*u* □ 0	□□ = 0
BC2f	Pinned	normal displacements restrained in-plane free rotation free	*w* = 0	*u* □ 0	□□ □ 0
BC3	Free edge	normal displacements free in-plane free rotation free	*w* □ 0	*u* □ 0	□□ □ 0
NOTE: on relationships between restraints at edges					

Editorial comment:- This table, drawn from EN 1993-1-6 does not work well for plate assemblies as v=0 applies to edges other than y=0 and y=b. It is certainly not appropriate for triangular plates.

EDGE SHEAR BOUNDARY CONDITIONS SHOULD BE INCLUDED FOR PLATES (DISPLACEMENTS IN THE PLANE OF THE PLATE PARALLEL TO THE EDGE)

6.4.4 Unstiffened plate segments

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

– the aspect ratio *b/a* of the plate is less than 0,5;

EDITORIAL COMMENT: Why 0,5?

– the plate is subjected to out of plane distributed loads which may be either linear or vary linearly;

– the strength, stability and stiffness of the frame or beam on which the plate segment is supported fulfil the assumed boundary conditions of the equivalent beam.

– EDITORIAL COMMENT: It might not be supported on a beam?

(2) 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.

(3) If an unstiffened plate is designed as an equivalent beam, its cross-section resistance should be checked for the combination of in-plane loading and out of plane loading effects with the design rules given in EN 1993-1-1.

(4) The stress resultants or stresses of an unstiffened subpanel should be verified against tensile rupture or plastic failure with the design rules given in

(5) If the first order deflection due to the out-of-plane loads is approximately congruent to the (plate) buckling mode due to in-plane compression forces, the interaction between both phenomena should be taken into account in buckling design (LS3), see 7.1.3 and 8.4.2. Supplementary rules for the design by simplified design method

6.4.5 Uni-directionally stiffened plate segments

(1) A plate segment that is stiffened in only one direction may be modelled as a series of adjacent simple beams, provided that the boundary conditions of the ends of each stiffened zone provide the required support.

(2) Where a stiffened plate segment is designed as an equivalent beam, as described in 5.2.3.4, the cross-section resistance and the buckling resistance of the equivalent beam should be checked for the combination of in-plane and out of plane loading effects using the interaction formula of 6.3.3 in EN 1993-1-1.

(3) The uni-directional stiffener acts integrally with the plate to make a T section beam, and the effective width of the plate acting with the stiffener should be evaluated using . . . eqns limiting the width and not exceeding the separation of the stiffeners.

(4) The effective width of the plate joined to an individual stiffener should be reduced to account for shear lag where . . .

(5) Where the plate does not meet the requirements of Formula 6.xx, the effects of shear lag should be included using the reduction factor β according to EN 1993-1-5 (give clause).

(6) To be continued

6.4.6 Bi-directionally stiffened plate segments

(1) A plate segment that is stiffened in both directions may be modelled as a grillage if it is regularly stiffened in the transverse and longitudinal direction.

(2) If a stiffened plate segment is modelled as a grillage as described in 5.2.3.4 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 6.3.3 of EN 1993-1-1.

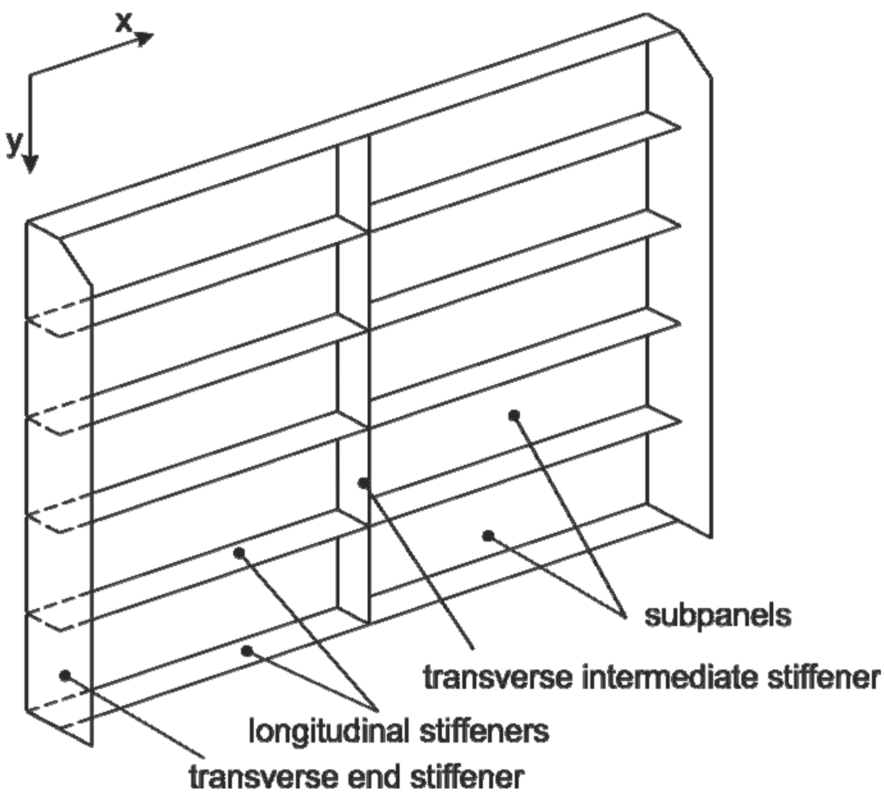


Figure 6.7 Typical rectangular stiffened plate

(3) 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 by the reduction factor β according to (give sub-clause) of EN 1993-1-5.

(4) For a member *i* of the grillage that is arranged in parallel to the direction of in-plane compression forces, the cross-sectional area A_i should also be determined taking account of the effective width of the adjacent subpanels due to plate buckling according to EN 1993-1-5.

(5) The interaction between shear lag effects and plate buckling effects, see Figure 6.8 should be considered by the effective area A_i (see Appendix A).

(6) The stresses in the stiffened plate are calculated as the superposition of membrane stresses due the the beam action of the stiffener within the effective width with the bending and membrane stresses due to bending in the subpanel between the stiffeners.

(6) If the stiffeners of a plate or a plate segment are only arranged in parallel to the direction of in-plane compression forces, the stiffened plate may be modelled as an equivalent beam on elastic springs, see EN 1993-1-5.

(6) If the stiffeners of a stiffened plate segment are positioned in the transverse direction to the compression forces, the interaction between the compression forces and bending moments in the unstiffened plate segments between the stiffeners should be verified according to 5.2.3.4.2(4).

(6) The longitudinal stiffeners should fulfil the requirements given in Clause 9 of EN 1993-1-5.

(7) The transverse stiffeners should fulfil the requirements given in Clause 9 of EN 1993-1-5.

6.5 Analysis by computer modelling

6.5.1 General

Text to be inserted here…see 7.2.4..

7 Ultimate limit states for plate assemblies

7.1 Types of ultimate limit states

(1)P Plates and plate assemblies shall be checked against the four ultimate limit states described in the following

7.1.1 LS1: Plastic failure resistance

(1)P For LS1, the design values of the actions shall be based on the most adverse relevant load combination (including the relevant γ_F and ψ *f*actors).

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

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

(4) The plastic reference load should be derived from a mechanism based on small deflection theory. The plastic failure limit state is defined as the condition in which a part of the structure either develops excessive plastic deformations, associated with development of a plastic mechanism, or suffers rupture.

(5) The limit state of tensile rupture should be taken as the condition in which a plate experiences tensile failure, leading to separation of the two parts of the plate or separation of two plate segments from each other at a junction.

(6) 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 Sub-Clause 8.2 in EN 1993-1-1 should be performed.

(7) 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).

NOTE The basic characteristic of this limit state is that the load or actions sustained (resistance) cannot be increased without exploiting a significant change in the geometry of the structure or strain-hardening of the material.

7.1.2 LS2: Cyclic plasticity

(1) The limit state of cyclic plasticity should be taken as the condition in which repeated cycles of loading and unloading produce yielding in tension or in compression or both at the same point, thus causing plastic work to be repeatedly done on the structure. This alternative yielding may lead to local cracking by exhaustion of the material's energy absorption capacity, and is thus 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 2.2) should be used for the calculation of the design stresses and stress resultants when checking LS2:

- expressions in Annex A or Annex C;
- elastic global analysis (LA or GNA);
- non-linear global analysis (MNA or GMNA) to determine the plastic strain range.
- To be continued

EDITORIAL NOTE: IT MIGHT BE MORE USEFUL TO LIST THE LIMIT STATES FOR WHICH ANALYSIS IS VALID IN TABLE 6.1 these sub-lists in 7 are then superfluous

7.1.3 LS3: Buckling

(1) The limit state of buckling should be taken as 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 plate segments. It can lead to inability to support the applied loads.

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

(3) For local plate buckling under membrane stresses, see EN 1993-1-5 in combination with the rules in 8.4, 9.4 and 10.4

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

Need to be clear here what is in scope – simple bifurcation analysis…

(5) 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 LS3:

- membrane and simple bending theory for simple geometries only (for exceptions, see relevant application parts of EN 1993)

– expressions in Annex A;

– linear elastic analysis (LA), which is a minimum requirement for stress analysis under general loading conditions (unless the load case is given in Annex A);

– linear elastic bifurcation analysis (LBA) is required for plate assemblies under general loading conditions if the critical buckling resistance is to be used (e.g. in an LBA-MNA global treatment);

– materially nonlinear analysis (MNA), which may be required for plate assemblies under general loading conditions if the plastic reference resistance is to be used (e.g. in an LBA-MNA global treatment);

– GMNIA, coupled with MNA, LBA and GMNA, using appropriate imperfections and calculated calibration factors.

(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.

7.1.4 LS4: Fatigue

(1) The limit state of fatigue is the condition in which repeated cycles of increasing and decreasing stress lead to the development and growth of a fatigue crack.

(2) The following methods of analysis (see 6.1) should be used for the calculation of the design stresses and stress resultants when checking LS4:

– expressions in Annex C, using stress concentration factors;

– elastic analysis (LA or GNA), using stress concentration factors.

(3) All variable actions that will be applied with more than N_f cycles in the design life time of the structure according to the relevant action spectrum in EN 1991, and in accordance with the appropriate application parts of EN 1993-3 or EN 1993-4, should be accounted for when checking LS4. The value of N_f should be taken as 10 000.

EDITORIAL NOTE: This was the value proposed in previous versions and no National Annex had different values.

7.2 Design concepts for the ultimate limit states design of plates and plate assemblies

7.2.1 General

(1) The limit state verification should be carried out using one of the following:

– stress design by plate and beam theoretical models;

– design by application of standard formulae;

– design by global numerical analysis (for example, by means of computer programs such as those based on the finite element method).

(2) The stress components should be placed in stress categories with different limits. Stresses that develop to meet equilibrium requirements (primary stresses) should be treated as more significant than stresses that are induced by the compatibility of deformations (secondary stresses).

NOTE The elastic-plastic material responses induced by different stress components in a plate assembly have different effects on the failure modes and the ultimate limit states. The requirements of equilibrium are essential, but those of compatibility can be ignored where appropriate.

(3) Local stresses caused by notch effects in construction details may be assumed to have a negligibly small influence on the resistance to static loading.

(4) The categories distinguished in the stress design should be primary, secondary and local stresses. Primary and secondary stress states may be replaced by stress resultants where appropriate.

(5) In a global analysis, the primary and secondary stress states should be replaced by the limit load and the strain range for cyclic loading.

(6) In general, it may be assumed that primary stress states control LS1. Further, it may be assumed that LS3 depends on primary stress states but may be affected by secondary stress states. The cyclic plasticity limit state LS2 depends on the combination of primary and secondary stress states, and local stresses govern LS4.

7.2.2 Stress design by plate- and beam-theoretical models

This seems to include computer analysis? Is that appropriate?

7.2.2.1 General

(1) Stresses in plate assemblies and stiffened panels may be calculated by analytical expressions:

- by using expressions derived from Kirchhoff-Love thin plate theory for individual plate segments or unstiffened plates; the expressions from Annex A.1 may be used for this purpose.
- by using expressions derived from orthotropic plate theory for uni- or bi-directionally stiffened plates and panels; the expressions in Annex A.2 may be used for this purpose
- by using simplified beam grillage models, combined with expressions for the plate bending effects in individual plates, for stiffened plates and panels.

(2) Where the stress design approach is used, the limit states should be assessed in terms of three categories of stress: primary, secondary and local. The categorisation is performed, in general, on the von Mises equivalent stress at a point, but buckling stresses cannot be assessed using this value.

(3) There are eight stress resultants at every point in a plate. In principle, these should all be calculated at each point of a plate assembly to determine their significance with respect to each limit state. However, the through-thickness shear stresses τ_{xz}, τ_{yz} due to the transverse shear forces q_{xz}, q_{yz} are insignificant compared with the other components of stress in almost all practical cases in plates, so they may usually be neglected in design.

(4) Accordingly, for most design purposes, the evaluation of the limit states may be made using only the six stress resultants at each point in a plate assembly n_x, n_y, n_{xy}, m_x, m_y, m_{xy}.

(5) If any uncertainty arises concerning the stress to be used in any of the limit state verifications, the von Mises equivalent stress at each surface of a plate should be used.

7.2.2.2 Primary stresses

(1) The primary stresses should be taken as the stress system required for equilibrium with the imposed loading. They may be calculated from any statically admissible determinate system. The plastic failure state (LS1) should be deemed to be reached when the primary stress reaches the yield

stress through the full thickness of the plate at a sufficient number of points, such that only the strain hardening reserve or a change of geometry would lead to an increase in the resistance of the structure.

(2) The calculation of primary stresses should be based on any system of stress resultants, consistent with the requirements of equilibrium of the structure. It may also take into account the benefits of plasticity theory. Alternatively, since linear elastic analysis satisfies equilibrium requirements, its predictions may also be used as a safe representation of the plastic failure state (LS1).

(3) Because limit state design for LS1 allows for full plastification of the cross-section at multiple locations, the primary stresses due to bending moments may be calculated on the basis of the plastic section modulus. Where there is interaction between stress resultants in the cross-section, interaction rules based on the Ilyushin yield criterion may be applied.

(4) The primary stresses should be limited to the design value of the yield strength.

7.2.2.3 Secondary stresses

(1) Secondary stresses are induced by internal compatibility and compatibility with the boundary conditions, and account should be taken of the stresses that are caused by imposed loading or imposed displacements (temperature, prestressing, settlement, shrinkage).

-(2) Where cyclic loading causes plasticity, and several loading cycles occur, consideration should be given to the possible reduction of resistance caused by the secondary stresses.

(3) Where the cyclic loading is of such a magnitude that yielding occurs both at the maximum load and again on unloading, account should be taken of a possible failure by cyclic plasticity associated with the secondary stresses.

(4) If the stress calculation is carried out using a linear elastic analysis that allows for all relevant compatibility conditions (effects at boundaries, junctions, variations in plate thickness etc.), the stresses that vary linearly through the thickness may be taken as the sum of the primary and secondary stresses and used in an assessment involving the von Mises yield criterion, see 6.2.

~~**NOTE:** The secondary stresses are never needed separately from the primary stresses.~~ wrong - LS2!

(4) The secondary stresses should be limited as follows:

- The sum of the primary and secondary stresses (including bending stresses) should be limited to $2f_{\mathrm{yd}}$ for the condition of cyclic plasticity (LS2: see Clause 7);
- The membrane component of the sum of the primary and secondary stresses should be limited by the design buckling resistance (LS3: see Clause 8).
- The sum of the primary and secondary stresses (including bending stresses) should be limited to the fatigue resistance (LS4: see Clause 9).

7.2.2.4 Local stresses

(1) Local stress raisers may lead to high localised stresses in a plate assembly due to notch effects (holes, welds, steps in thickness, attachments, and joints). These should be taken into account in a fatigue assessment (LS4).

(2) For construction details given in EN 1993-1-9, the fatigue design may be based on the nominal linear elastic stresses (the sum of the primary and secondary stresses) at the relevant point. For all other details, the local stresses may be calculated by applying stress concentration factors (notch factors) to the stresses calculated using a linear elastic stress analysis.

(3) The local stresses should be limited according to the requirements for fatigue (LS4) set out in Clause 9.

7.2.3 Design using standard formulae

(1) Where design using standard formulae is used, the limit states may be represented by standard expressions for maximum loading scenarios, which have been derived for either membrane and bending analysis, plastic mechanism theory or linear elastic analysis.

(2) The membrane and bending analysis expressions given in Annex A may be used to determine the primary stresses needed for assessing LS1 and LS3. The LS3 assessment may be based on the membrane part of these expressions.

(3) The expressions for plastic design given in Annex B, which are plastic reference resistances, may be used to obtain plastic failure loads needed for assessing LS1.

(4) The expressions for linear elastic plate bending analysis (LA) given in Annex C may be used to determine the combinations of primary and secondary stresses needed to assess LS2 and LS4.

7.2.4 Design by global numerical analysis

(1) Where a global numerical analysis is used, the assessment of the limit states should be carried out using one of the alternative types of analysis specified in 6.1 (with the exception of membrane and simplified bending analysis) applied to the complete structure.

(2) Linear elastic analysis (LA) may be used to determine stresses or stress resultants, for use in assessing LS2 and LS4. The membrane parts of the stresses found by LA may be used in assessing LS3. LS1 may be assessed using LA, but LA only gives an approximate estimate and its results should be interpreted as set out in Clause 6.

(3) Linear elastic bifurcation analysis (LBA) may be used to determine the critical buckling resistance of the structure, for use in assessing LS3.

(4) A materially nonlinear analysis (MNA) may be used to determine the plastic reference resistance, and this may be used for assessing LS1. Under a cyclic loading history, an MNA analysis may be used to determine plastic strain incremental changes, for use in assessing LS2. The plastic reference resistance is also required as part of the assessment of LS3, and this may be found from an MNA analysis.

(5) Geometrically nonlinear elastic analysis (GNA) includes consideration of the deformations of the structure, but none of the design methodologies of Clause 8 permit these to be used without a GMNIA analysis.

(6) Geometrically and materially nonlinear analysis (GMNA and GMNIA) may be used to determine collapse loads for the perfect (GMNA) and the imperfect structure (GMNIA). The GMNA analysis may be used in assessing LS1, as detailed in 6.3. The GMNIA collapse load may be used, with additional consideration of the GMNA collapse load, for assessing LS3 as detailed in 8.7. Under a cyclic loading history, the plastic strain incremental changes taken from a GMNA analysis may be used to assess LS2.

8 Ultimate limit state design of unstiffened plates

8.1 General

(1)P Unstiffened plates and subpanels between stiffeners of stiffened 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 global numerical design.

8.2 Plastic failure limit state (LS1)

8.2.1 Stress-based design

8.2.1.1 Design values of stresses

(1) Design values of stress may be calculated from membrane and bending stress resultants obtained from standard formulae, such as the ones given in Annex A.1.

(2) In case of a two dimensional stress field resulting from a membrane and simplified bending analysis or a linear elastic analysis, the equivalent von Mises stress $\sigma_{eq,Ed}$ may be determined, as follows:

$$\sigma_{eq,Ed} = \sqrt{\sigma^2_{x,Ed} - \sigma_{x,Ed}\,\sigma_{y,Ed} + \sigma^2_{y,Ed} + 3\tau^2_{xy,Ed}} \qquad (8.1)$$

where:

$$\sigma_{x,Ed} = \frac{n_{x,Ed}}{t} \pm \frac{m_{x,Ed}}{t^2/4} \qquad \sigma_{y,Ed} = \frac{n_{y,Ed}}{t} \pm \frac{m_{y,Ed}}{t^2/4} \qquad (8.2)$$

$$\tau_{xy,Ed} = \frac{n_{xy,Ed}}{t} \pm \frac{m_{xy,Ed}}{t^2/4} \qquad (8.3)$$

and $n_{x,Ed}$, $n_{y,Ed}$, $n_{xy,Ed}$, $m_{x,Ed}$, $m_{y,Ed}$ and $m_{xy,Ed}$ are defined in **Error! Reference source not found.**(1) and (2).

NOTE The above expressions give a simplified conservative equivalent stress for design

8.2.1.2 Design values of resistance

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

$$n_{xRd} = n_{yRd} = \frac{1{,}155\, f_{y,d}\, t}{\gamma_{M0}} \qquad (8.4)$$

(2) The membrane shear yield resistance should be taken as

$$n_{xyRd} = \frac{0{,}577\, f_{y,d}\, t}{\gamma_{M0}} \qquad (8.5)$$

(3) 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

$$m_{Rd} = \frac{1{,}16\, f_{y,d} t^2}{\gamma_{M0}}$$

(4) Where membrane tension exceeds 10% of the tensile yield resistance, the reduced bending resistance in a yield line should be taken as

Revised and corrected equations are needed here

$$m_{Rd} = \frac{1{,}16\, f_{y,d} t^2}{\gamma_{M0}} \left\{ 1 - n_{eq}^2 \right\}$$

where:

$$n_{eq} = \frac{\gamma_{M0} n_{vM,E,d}}{n_{xRd}}$$

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

EDITORIAL NOTE JMR THINKS THAT THE TEXT BELOW IS TOO SIMPLE AND CONSERVATIVE AND SHOULD BE REPLACED BY SOMETHING VERY SIMILAR TO THE TEXT ABOVE THAT HAD BEEN DELETED.

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

$$\sigma_{eq,Ed} \leq f_{yk} / \gamma_{M0} \tag{8.6}$$

(6) The partial factor for resistance γ_{M0} should be taken from the relevant application standard.

(7) Where no application standard exists for the form of construction involved, or the application standard does not define the relevant values of γ_{M0}, the value of γ_{M0} should be taken from EN 1993-1-1.

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

8.2.2 Design using standard formulae

(1) For the design of unstiffened plates and plate assemblies of standard shape and subjected only to standard load cases, the rules given in Annex A may be applied.

8.2.3 Design by global analysis

8.2.3.1 Linear-elastic global analysis

(1) Where the internal stresses in a plate assembly are determined by a global numerical analysis which is based on a materially linear analysis, the maximum equivalent von Mises stress $\sigma_{eq,Ed}$ of the plate assembly should be calculated from the stress resultants using Formula (8.1).

EDITORIAL NOTE: THIS IS NOT A GOOD IDEA, AS IT WILL LEAD TO GLOBAL NUMERICAL ANALYSIS BEING MORE CONSERVATIVE AND DEMANDING THAN THE SIMPLE HAND CALCULATIONS, WHICH IS CONTRARY TO THE PRINCIPLE THAT THE MORE CAREFUL THE ANALYSIS, AND THE MORE EXPENSE ON DOING IT, THE MORE ECONOMIC OR SAFER THE STRUCTURE WILL BE

(2) The resistance should be assessed using .. ? To be continued

8.2.3.2 Nonlinear global analysis

(1) To be continued .... MNA /GMNA

(2) ...

(3) ....

8.3 Cyclic plasticity limit state (LS2)

8.3.1 General

(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 by either of the following design approaches:

- When using a design method based on notional linear-elastic stresses, consisting of both primary and secondary stress components, the sum of primary and secondary elastic stresses may exceed the yield stress provided that the primary stresses do not lead to yielding.

EDITORIAL COMMENT: ANY YIELDING ANYWHERE IN THE PLATE?

- When using a design method based on the determination of the sequence, number of occurrences and accumulation of total plastic and elastic strains, non-linear kinematic or Chaboche-type hardening laws may be considered to verify the occurrence of ratcheting under repeated loading.

EDITORIAL COMMENT: BIBLIOGRAPHIC ITEM NEEDED FOR A STATEMENT LIKE THIS

EDITORIAL COMMENT: FOR EASE OF USE, IT IS IMPORTANT TO GIVE THE DESIGNER A SIMPLER TREATMENT AS THE REFERENCE METHOD, EVEN IT IS MORE CONSERVATIVE THAN THE ABOVE

(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 computer 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

....check and perhaps replace this sub-clause with Bree diagram & similar methods from pressure vessel codes

(1) The plate assembly should be analysed using an LA or GNA analysis of the structure subject to 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 $\Delta\sigma_{x,Ed,i}$, $\Delta\sigma_{\theta,Ed,i}$, $\Delta\tau_{x\theta,Ed,i}$ 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:

$$\Delta\sigma_{eq,Ed,i} = \sqrt{\Delta\sigma^2_{x,Ed,i} - \Delta\sigma_{x,Ed,i} \cdot \Delta\sigma_{\theta,Ed,i} + \Delta\sigma^2_{\theta,Ed} + 3\Delta\tau^2_{x\,\theta,Ed,i}} \quad \text{... (8.7)}$$

(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 plate assembly segments, 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 segment (as opposed to the value calculated at the intersection of the two middle surfaces).

NOTE This allowance is relevant where the stress changes very rapidly close to the junction.

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

$$\Delta\sigma_{Ed} \leq \Delta\sigma_{Rd} \quad (8.8)$$

where:

$\Delta\sigma_{Ed}$ is the largest value of the von Mises equivalent stress range, and

$$\Delta\sigma_{eq,Ed} = \sqrt{\Delta\sigma^2_{x,Ed} + \Delta\sigma^2_{y,Ed} - \Delta\sigma_{x,Ed}\,\Delta\sigma_{y,Ed} + 3\Delta\tau^2_{Ed}} \quad (8.9)$$

at the relevant point of the plate segment due to the relevant combination of design actions.

(5) In a materially linear design the resistance of a plate segment against cyclic plasticity / low cycle fatigue may be verified by the von Mises stress range limitation $\Delta\sigma_{Rd}$.

$$\Delta\sigma_{Rd} = 2{,}0\, f_{yk} / \gamma_{M0} \quad (8.10)$$

NOTE For the numerical value of γ_{M0} see EN 1993-1-1 or the relevant application standards.

8.3.3 Design by global analysis – accumulated strains

.... check and possibly replace - strain accumulation = ratcheting check is incorrect if done with ideal-plastic material

(1) Where a materially nonlinear computer analysis is carried out, the plate should be subject to the design values of the actions.

(2) The total accumulated von Mises equivalent 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.

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

$$\varepsilon_{eq,Ed} = m \, \Delta\varepsilon_{eq,Ed} \tag{8.11}$$

where:

m is the number of cycles in the design life;

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

(4) 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

$$\varepsilon_{p,eq.Ed} \le n_{eq} \frac{f_{yk}}{E\gamma_{M0}} \tag{8.12}$$

where the value n_{eq} = 25 should be adopted unless a more thorough treatment can be justified.

NOTE The value of γ_{M0} should be taken from the relevant application standard

8.4 Buckling limit state (LS3)

8.4.1 General

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

(2) Individual unstiffened plates and subpanels between stiffeners can fail in buckling if the compressive membrane strains exceed the buckling resistance.

(3) The increase of pre-buckling out-of-plane deformations caused by transversal loads acting on the plate surface should be considered in the design against buckling.

(4) The design may be performed through the use of buckling formulae which account for the effect of loads and imperfections in direction transverse to the plate central plane, or by numerical analysis.

8.4.2 Design using buckling formulae

(1) ….. It is very difficult to write any buckling rules here that are relevant to storage structures with unstiffened plates. The vertical compression from frictional drag of solids is far too small to be important in buckling here. The problem of local supports is the only viable case for buckling, but it involves very local buckles in a plate where the stress distribution is varying very rapidly as a combination of shear, horizontal tension and some vertical compression. This is so complicated as to be way beyond the scope of EN 1993-1-7 and has to be a detail in EN 1993-4-1.

There seems to be no point in writing rules about plate buckling that correspond to the rules of EN 1993-1-5. The best solution wouuld appear to be to refer to EN 1993-1-5 for all cases where there is significantly uniform compression and to leave the rest to EN 1993-4-1.

(2) …. IT IS IMPORTANT THAT IF A BUCKLING CURVE IS TO BE PLACED HERE, IT SHOULD USE A CAPACITY CURVE THAT MATCHES EN 1993-1-6 SINCE THE STRESS STATE IS CERTAINLY NOT UNIAXIAL HERE AND FORMULATIONS LIKE THE WINTER CURVE AND THOSE OF EN 1993-1-5 ARE NOT HELPFUL , modified winter formula

(3) … To be continued

(4) If the simplified analysis methods for unstiffened plates in 6.4.4 is used and a case where the situation as described in 6.4.4(3) is present, the interaction formula specified in 8.3.3 of EN 1993-1-1, may be applied to an equivalent beam cross-section.

8.4.3 Design by global numerical analysis

(1) If a numerical analysis is used for the verification of buckling, the effects of imperfections should be taken into account. These imperfections may be:

geometrical imperfections:

- deviations from the nominal geometric shape of the plate (initial deformation, out of plane deflections);
- irregularities of welds (minor eccentricities);
- deviations from nominal thickness.

material imperfections:

- residual stresses because of rolling, pressing , welding, straightening;
- non-homogeneities and anisotropies.

(2) The geometrical and material imperfections should be taken into account by an initial equivalent geometric imperfection of the perfect plate. The shape of the initial equivalent geometric imperfection should be derived from the relevant buckling mode.

FIGURE MISSING HERE: IS IT RELEVANT ANYWAY?

Figure 8.1 Initial equivalent geometric bow imperfection e_0 in a plate segment

(3) As a conservative assumption the amplitude may be taken as $e_0 = a/200$ where $b \leq a$.

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

(5) In all cases the reliability of a numerical analysis should be checked with known results from tests or benchmark analysis cases.

8.5 Fatigue limit state (LS4)

8.5.1 General

(1) The fatigue assessment presented in EN 1993-1-9 should be used, except as provided here.

(2) The partial factor for resistance to fatigue γ_{Mf} should be taken from the relevant application standard.

(3) which stresses, which FAT classes? To be continued

9 Ultimate limit state design of uni-directionally stiffened panels

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 numerical design.

9.2 Plastic failure limit state (LS1)

9.2.1 Stress-based design

(1) To be continued

9.2.2 Design using standard formulae

(1) To be continued

9.2.3 Design by global analysis

(1) To be continued

9.3 Cyclic plasticity limit state (LS2)

9.3.1 General

(1) To be continued

9.3.2 Stress-based design

(1) To be continued

9.3.3 Design by global analysis – accumulated strains

(1) To be continued

9.4 Buckling limit state (LS3)

9.4.1 General

(1) To be continued

9.4.2 Design using using buckling formulae

(1) To be continued

(2) To be continued

(3) If the simplified analysis method of 6.4.5 is used, the buckling verification of a member *i* of the grillage may be performed using the interaction formula in 6.3.3 of EN 1993-1-1, taking into account the following loading conditions:

- effects of out of plane loadings;
- equivalent axial force in the cross section A_i (see **Error! Reference source not found.**) due to normal stresses in the plate;
- eccentricity *e* of the equivalent axial force N_{Ed} with respect to the centre of gravity of the cross-sectional area A_i.

9.4.3 Design by global numerical analysis

(1) To be continued

(2) To be continued

9.5 Fatigue limit state (LS4)

9.5.1 General

(1) To be continued

10 Ultimate limit state design of bi-directionally stiffened panels (grillages)

10.1 General

(1)P Orthogonally stiffened plates (grillages) 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 numerical design.

10.2 Plastic failure limit state (LS1)

10.2.1 Stress-based design

(1) To be continued

10.2.2 Design using standard formulae

(1) To be continued

10.2.3 Design by global analysis

(1) To be continued

10.3 Cyclic plasticity limit state (LS2)

10.3.1 General

(1) To be continued

10.3.2 Stress-based design

(1) To be continued

10.3.3 Design by global analysis – accumulated strains

(1) To be continued

10.4 Buckling limit state (LS3)

10.4.1 General

(1) To be continued

10.4.2 Design using buckling formulae

(1) To be continued

10.4.3 Design by global numerical analysis

(1) To be continued

10.5 Fatigue limit state (LS4)

10.5.1 General

(1) To be continued

10.5.2 Other provisions

(1) To be continued

11 Serviceability limit state

11.1 General

(1) The principles for serviceability limit state given in Clause 7 of EN 1993-1-1 should also be applied to plates and plate assemblies.

(2) For plate assemblies especially the limit state criteria given in 11.2 and 11.3 should be verified.

11.2 Out of plane deflection

(1) The limit of the out of plane deflection w should be defined as the condition in which the effective use of a plate segment is ended.

NOTE For limiting values of out of plane deflection w see application standard.

11.3 Excessive vibrations

(1) Excessive vibrations should be defined as the limit condition in which either the failure of a plate assembly occurs by fatigue caused by excessive vibrations of the plate or serviceability limits apply.

NOTE For limiting values of slenderness to prevent excessive vibrations see application standard.

(2) To be continued

SOME POSSIBLY USEFUL CLAUSES LEFT OVER FROM THE PREVIOUS STANDARD ARE TEMPORARILY PLACED HERE

11.4 Buckling resistance

11.4.1 General

(1) If a plate segment of a plate assembly is loaded by in-plane compression or shear, its resistance to plate buckling should be verified with the design rules given in EN 1993-1-5.

(2) Flexural, lateral torsional or distortional stability of the stiffness should be verified according to EN 1993-1-5, see also 5.2.3.4 (8) and (9)

(3) For the interaction between the effects of in-plane and out of plane loading, see Clause 8.

11.4.2 Supplementary rules for the design by global analysis.

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

$$F_{Ed} \leq F_{Rd} \tag{11.1}$$

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

$$F_{Rd} = k\, F_{Rk}/\gamma_{M1} \tag{11.2}$$

where:

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

k is the calibration factor, see (6).

NOTE For the numerical value of γ_{M1} see 1.1(2).

(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 taking into account the design values of the relevant combination of actions F_{Ed}. In addition, the analysis should take into account the imperfections as defined in XXXX.

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

– maximum load of the load-deformation-curve (limit load);

– maximum tolerable deformation in the load deformation curve before reaching the bifurcation load or the limit load, if relevant.

(5) The reliability of the numerically determined buckling resistance should be checked in one of the following ways (see 8.8 in EN 1993-1-6):

a) by calculating other plate buckling cases, for which characteristic buckling resistance values $F_{Rk,known}$ are known, with similar imperfection assumptions. The check cases should be similar in their buckling controlling parameters (e.g. non-dimensional plate slenderness, post-buckling behaviour, imperfection-sensitivity, material behaviour);

b) by comparison of calculated values with test results FRk,known.

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

$$k = F_{Rk,known} / F_{Rk.check} \tag{11.3}$$

where:

$F_{Rk,known}$ is a known characteristic value;

$F_{Rk.check}$ is the calculation outcome for the known case.

This description is incomplete, but it would be better to refer to EN 1993-1-6 for this whole process.

11.4.3 Supplementary rules for the design by simplified design methods

(1) If a stiffened plate segment is subdivided into subpanels and equivalent effective stiffeners as described in 5.2.3.4 the buckling resistance of the stiffened plate segment may be checked with the design rules given in EN 1993-1-5. Lateral buckling of free stiffener-flanges may be checked according to 6.3.3 in EN 1993-1-1.

(2) The buckling resistance of the equivalent effective part of the plate, which is defined in 5.2.3.4 , that acts with the stiffener may be checked using the design rules given in EN 1993-1-1.

Annex A [informative] Membrane and simple elastic bending analysis stress resultants in individual plates and plate assemblies

A.1 General

Scope:

Different geometries: square, rectangular, triangular, trapezoidal

Different load cases: uniform pressure, linear variation, Janssen variation

Different boundary conditions where deemed appropriate

A.2 Symbols

A.3 Simple equations for SMBT treatment of rectangular plate assemblies

A.4 Simple equations for SMBT treatment of non-rectangular plate assemblies

Annex B [informative] Expressions for plastic reference resistances in individual plates and plate assemblies

B.1 General

Scope:

Different geometries: square, rectangular, triangular, trapezoidal

Different load cases: uniform pressure, linear variation, Janssen variation

Different boundary conditions where deemed appropriate

B.2 Symbols

B.3 Rectangular plates under uniform pressure

B.4 Non-rectangular plates under uniform pressure

B.5 Rectangular plates under hydrostatic pressure

B.6 Non-rectangular plates under hydrostatic pressure

B.7 Rectangular plates under Janssen pressures

B.8 Non-rectangular plates under Janssen pressures

Annex C [informative] Expressions for linear elastic stresses in rectangular plates from small deflection theory

CONTENT TO BE EXTENDED AND THE EXISTING DATA ALSO VERIFIED

C.1 General

(1) This annex provides design formulae for the calculation of internal stresses in unstiffened rectangular 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.

(2) Design formulae are provided for the following load cases:

– uniformly distributed loading on the entire plate, see C.3;

– central patch loading distributed uniformly over a patch area, see C.4.

(3) The maximum deflection w and the maximum bending stresses σ_{bx} and σ_{by} in a plate segment may be calculated with the coefficients given in the tables in C.3 and C.4. The coefficients use a Poisson's ratio ν of 0,3.

C.2 Symbols

(1) The symbols used are:

p_{Ed} is the design value of the distributed load;

~~p_{Ed} is the design value of the patch loading;~~

The notation here should be consistent with EN 1991-4. In EN 1993-1-7 *q* is the transverse shear force in a plate, which is important in plate assemblies in transferring forces between plate segments So the notation *p* should be the loading, consistently with EN 1991-4.

~~a is the smaller side of the plate;~~

~~b is the longer side of the plate;~~

a is the longer side of the plate;

b is the smaller side of the plate;

NOTE THAT THESE MUST BE DEFINED AS AMENDED HERE

CONVENTIONAL TREATMENT IS *a* IS THE LONG SIDE, *b* IS THE SHORT SIDE (EN 1993-1-1)

t is the uniform thickness of the plate;

E is the elastic modulus;

k_w is the coefficient for the maximum deflection of the plate appropriate to the boundary conditions of the plate specified in the data tables;

$k_{\sigma bx}$ is the coefficient for the maximum bending stress σ_{bx} in the plate appropriate to the boundary conditions of the plate specified in the data tables;

$k_{\sigma by}$ is the coefficient for the maximum bending stress σ_{by} in the plate appropriate to the boundary conditions of the plate specified in the data tables.

C.3 Uniformly distributed loading

C.3.1 Out of plane deflection

(1) The maximum deflection w in a plate segment which is loaded by uniformly distributed loading may be calculated as follows:

$$w = k_{\mathrm{w}} \frac{p_{\mathrm{Ed}}\, a^4}{E\, t^3} \tag{C.1}$$

NOTE Expression (C.1) is only valid where w is small compared with the thickness t because changes of geometry make the plate stiffer as the deformations grow.

C.3.2 Internal stresses

(1) The bending stresses σ_{bx} and σ_{by} in a rectangular plate segment may be determined using the following equations:

$$\sigma_{\mathrm{bx,Ed}} = k_{\sigma\mathrm{bx}} \frac{p_{\mathrm{Ed}}\, \mathrm{a}^2}{t^2} \tag{C.2}$$

$$\sigma_{\mathrm{by,Ed}} = k_{\sigma\mathrm{by}} \frac{p_{\mathrm{Ed}}\, \mathrm{a}^2}{t^2} \tag{C.3}$$

(2) For a plate segment, the von Mises equivalent stress may be calculated by using the bending stresses given in (1) in Formula (C.4):

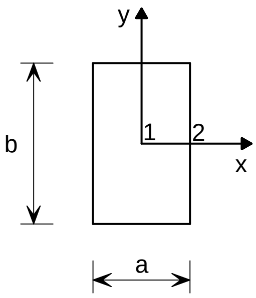
$$\sigma_{\mathrm{eq},Ed} = \sqrt{\sigma^2_{\mathrm{bx},Ed} + \sigma^2_{\mathrm{by},Ed} - \sigma_{\mathrm{bx},Ed}\,\sigma_{\mathrm{by},Ed}} \tag{C.4}$$

NOTE The points for which the state of stress are defined in the data tables are located either on the centre lines or on the boundaries, so that due to symmetry of the assumed boundary conditions, the bending shear stresses τ_b are zero.

ALL THE TABLES IN THE PUBLISHED EN 1993-1-7 (2007) WILL BE REPLACED BY ALGEBRAIC EXPRESSIONS THAT CLOSELY APPROXIMATE THE CORRECT RESULT, BUT GIVE CONTINUOUS VARIATIONS WITH THE GEOMETRIC PARAMETERS FOR EASY ADOPTION INTO SPREADSHEETS

C.3.3 Coefficients k for uniformly distributed loadings

Table C.1: Coefficients k

CHANGE DRAWING TO CORRECT a and b 	**Loading:** Uniformly distributed loading
	Boundary conditions: All edges are rigidly supported and rotationally free

$$k_{w1} = 0.178 - 0.135(b/a)$$

$$k_{\sigma bx1} = 0.925 - 0.643(b/a)$$

$$k_{\sigma by1} = 0.153 + 0.295(b/a) - 0.018(b/a)^2 - 0.144(b/a)^3$$

Simply supported rectangular plates uniform pressure

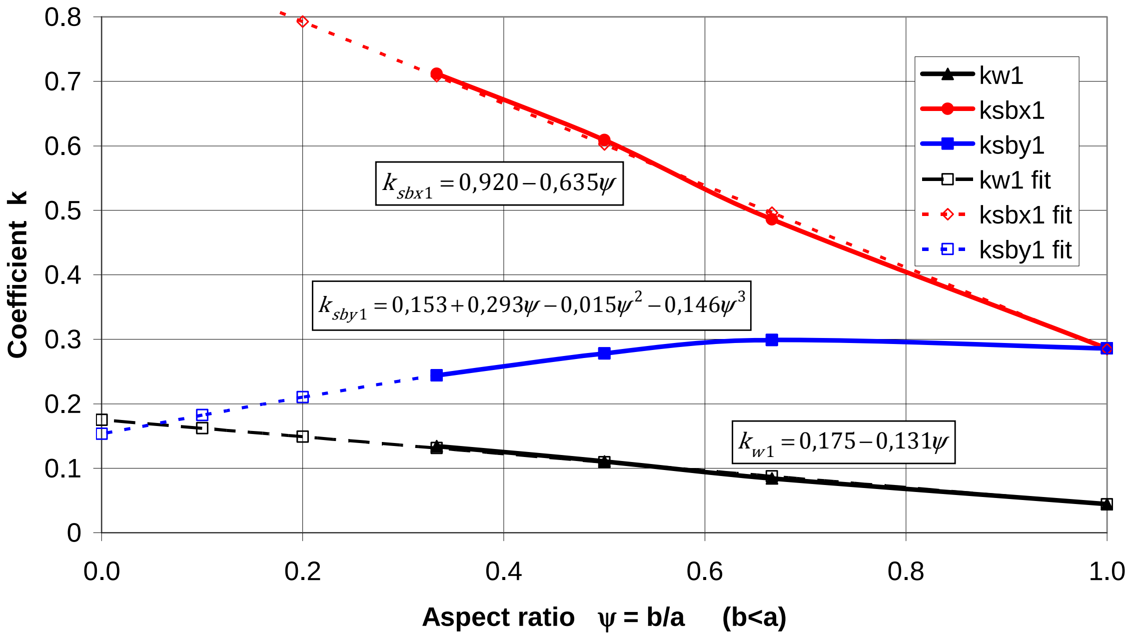
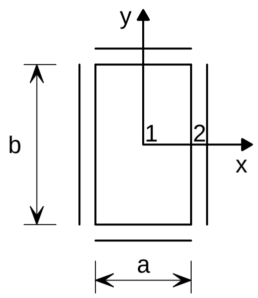


Table C.2: Coefficients k

CHANGE DRAWING TO CORRECT a and b 	**Loading:** Uniformly distributed loading
	Boundary conditions: All edges are rigidly supported and rotationally fixed.

$$k_{w1} = 0{,}039 - 0{,}025\psi$$

$$k_{sbx1} = 0{,}224 + 0{,}159\psi - 0{,}247\psi^2$$

$$k_{sby1} = 0{,}011 + 0{,}222\psi - 0{,}097\psi^2$$

$$k_{sbxs} = -0{,}458 - 0{,}294\psi + 0{,}444\psi^2$$

Built in rectangular plates uniform pressure

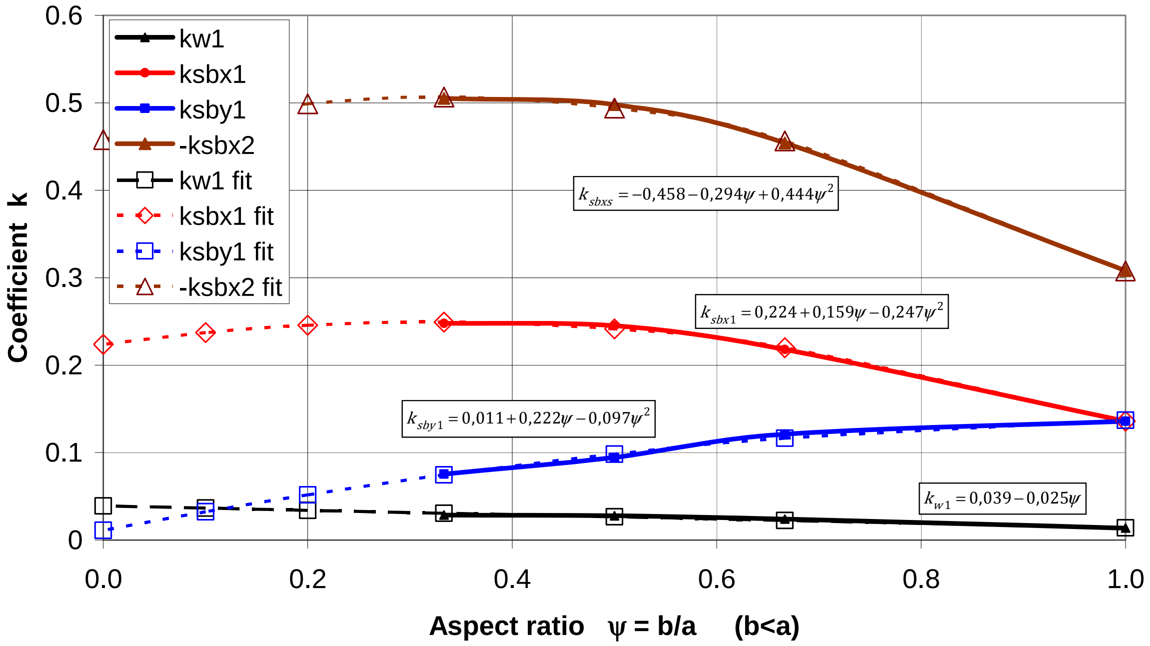
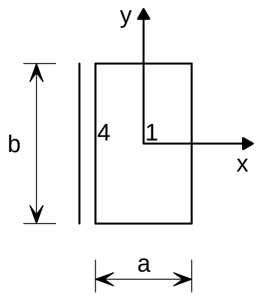


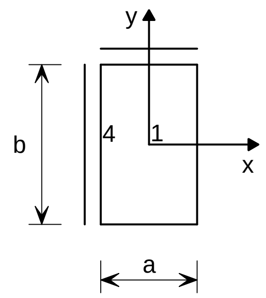
Table C.3: Coefficients k

CHANGE DRAWING TO CORRECT a and b 	**Loading:** Uniformly distributed loading
	Boundary conditions: Three edges are rigidly supported and rotationally free and one edge is rigidly supported and rotationally fixed.

NOT ENOUGH DATA HERE TO PRODUCE A RELIABLE EQUATION

CALCULATIONS WILL BE UNDERTAKEN TO GIVE SIMPLE EQUATIONS FOR THE SECOND DRAFT

Table C.4: Coefficients k

<table>
<tr><td rowspan="2">CHANGE DRAWING TO CORRECT a and b

</td><td>Loading:
Uniformly distributed loading</td></tr>
<tr><td>Boundary conditions:
Two edges are rigidly supported and rotationally free and two edges are rigidly supported and rotationally fixed.</td></tr>
</table>

$$k_{w1} = 0{,}084 - 0{,}059\psi$$

$$k_{sbx1} = 0{,}529 - 0{,}344\psi$$

$$k_{sby1} = -0{,}034 + 0{,}525\psi - 0{,}306\psi^2$$

$$k_{sbx4} = -0{,}999 + 0{,}623\psi$$

SS & BI mixed rectangular plates Case C.4 uniform pressure

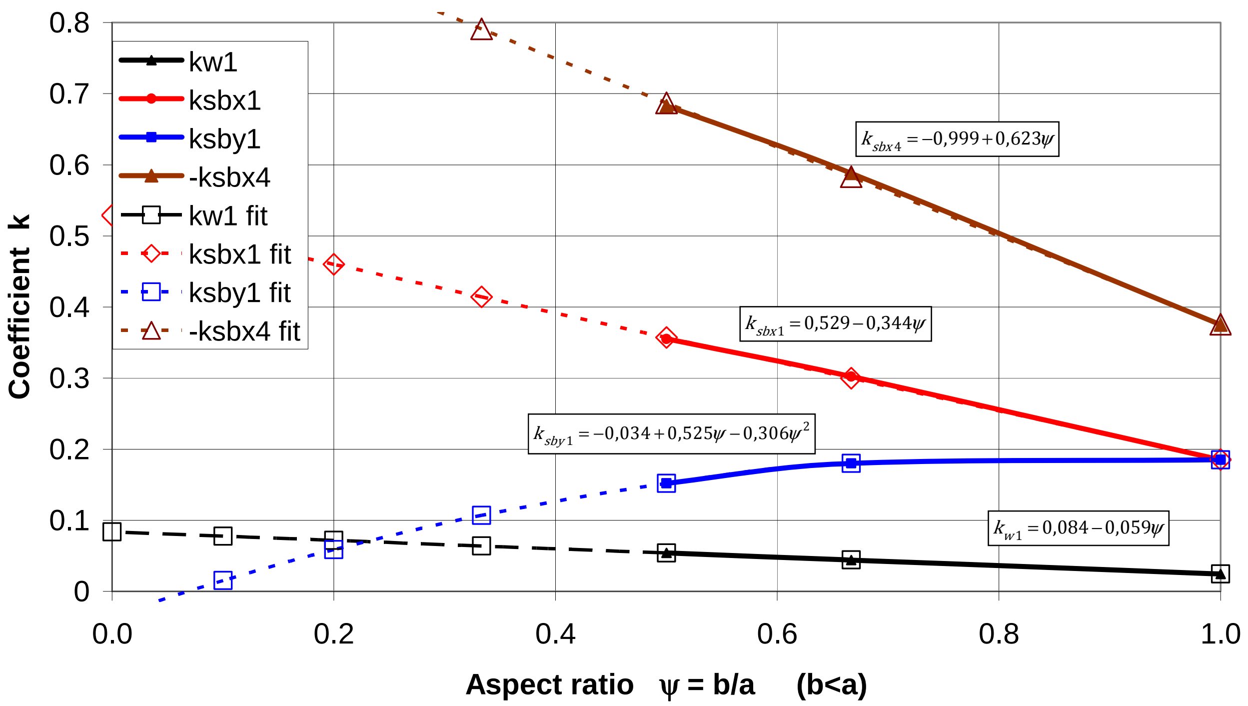
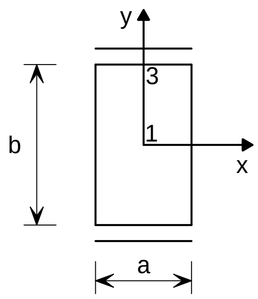


Table C.5: Coefficients k

CHANGE DRAWING TO CORRECT a and b 	**Loading:** Uniformly distributed loading **Boundary conditions:** Two opposite short edges are clamped, the other two edges are simply supported.

$$k_{w1} = 0{,}257 - 0{,}424\psi + 0{,}187\psi^2$$

$$k_{sbx1} = 1{,}312 - 2{,}003\psi + 0{,}837\psi^2$$

$$k_{sby1} = 0{,}200 + 0{,}339\psi - 0{,}342\psi^2$$

$$k_{sby3} = -0{,}906 + 0{,}271\psi + 0{,}216\psi^2$$

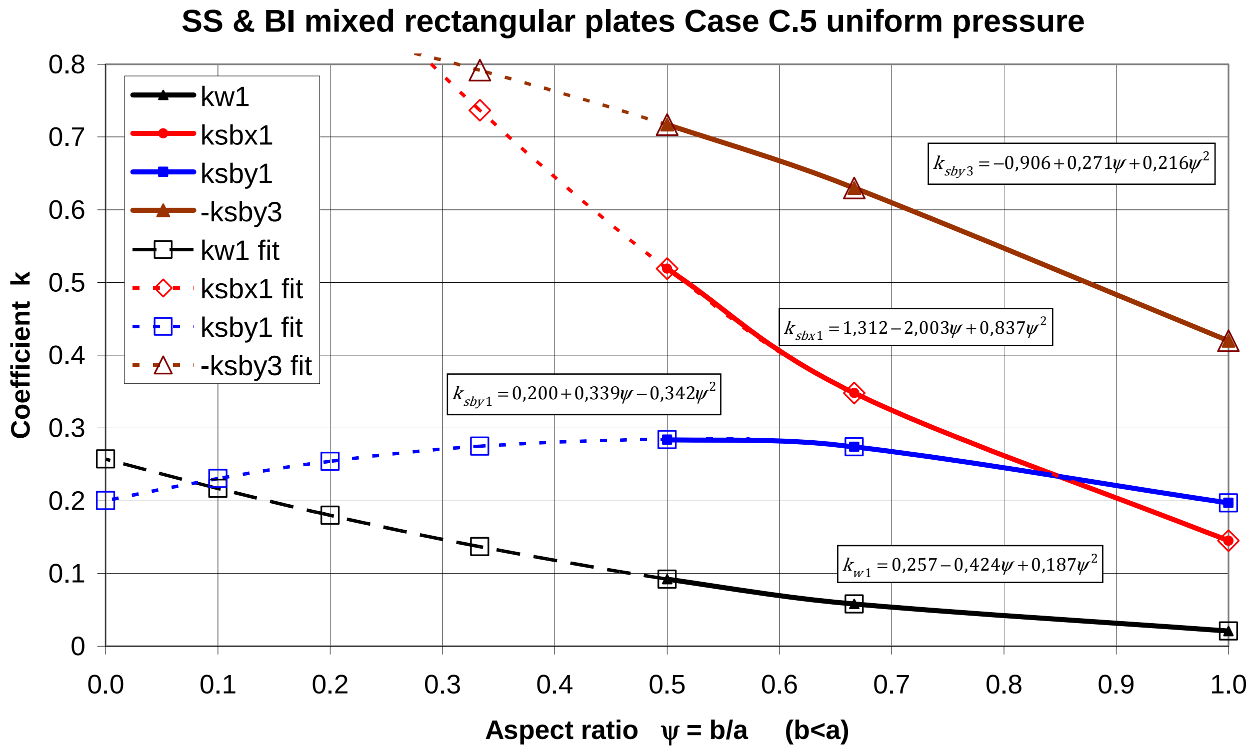
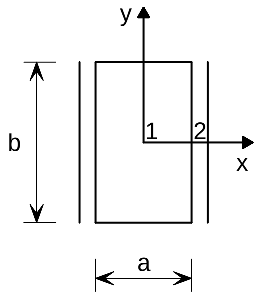


Table C.6: Coefficients k

CHANGE DRAWING TO CORRECT a and b



Loading:

Uniformly distributed loading

Boundary conditions:

Two opposite long edges are clamped, the other two edges are simply supported.

b/a	k_{w1}	$k_{\sigma bx1}$	$k_{\sigma by1}$	$k_{\sigma bx2}$
1,5	0,02706	0,240	0,106	-0,495
2,0	0,02852	0,250	0,0848	-0,507

NOT ENOUGH DATA HERE TO PRODUCE A RELIABLE EQUATION

C.4 Linearly varying pressure

Expressions here for plates under pressure that is constant in one direction and varies linearly in the other (hydrostatic)

C.4 Janssen pressure variations

Expressions here for plates under a Janssen vertical distribution but a hyperbolic horizontal variation as required for rectangular silo design.

These will be related to the constant and hydrostatic values given above.

Annex D [informative] Assessment of effective area of a stiffened plate

$$A_i = [\rho_c\ (A_{L,eff} + \Sigma\, \rho_{pan,i}\, b_{pan,i}\, t_{pan,i}\ )]\ \beta^{\kappa} \quad \text{(D.1)}$$

where:

$A_{L,eff}$ is the effective area of the stiffener considering to local plate buckling of the stiffener;

ρ_c is the reduction factor due to global plate buckling of the stiffened plate segment, as defined in 4.5.4(1) of EN 1993-1-5;

$\rho_{pan,i}$ is the reduction factor due to local plate buckling of the subpanel *i*, as defined in 4.4(1) of EN 1993-1-5;

$b_{pan,i}$ is the width of the subpanel *i*, as defined in 4.5.1(3) of EN 1993-1-5;

$t_{pan,i}$ is the thickness of the subpanel *i*;

β is the effective width factor for the effect of shear lag, see 3.2.1 of EN 1993-1-5;

κ is the ratio defined in 3.3 of EN 1993-1-5.

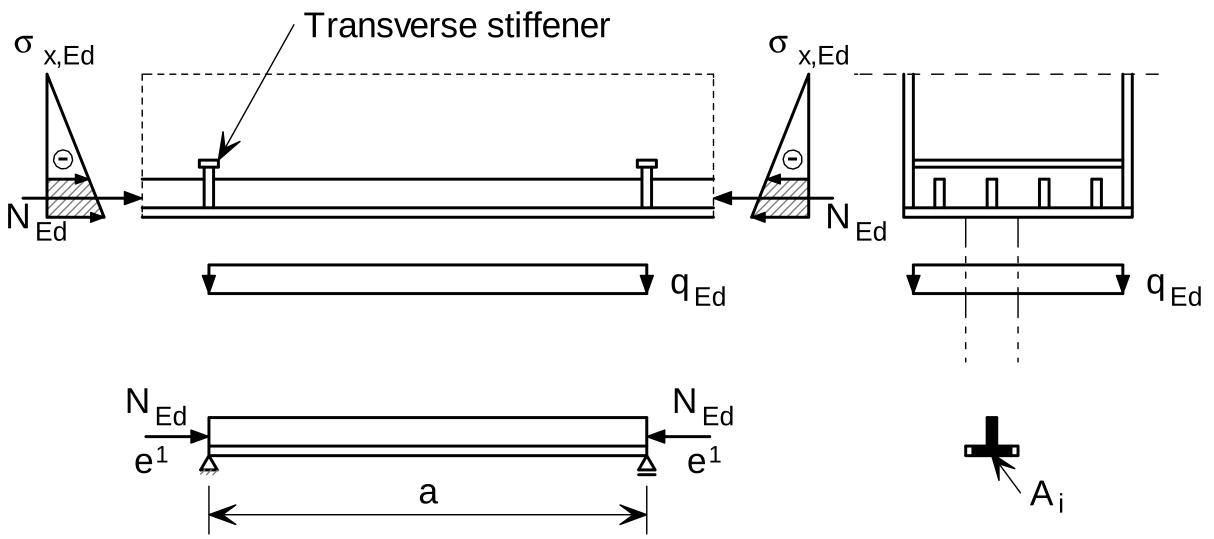


Figure D.1 Definition of the cross-section A_i

EDITORIAL COMMENT: THIS IMAGE LOOKS LIKE A BRIDGE GIRDER. THERE IS NO INDICATION THAT THE LOADING IS PREDOMINANTLY TRANSVERSE, WHICH IS WHAT THIS STANDARD MUST COVER. MORE DESCRIPTION OR MORE FIGURES ARE NEEDED TO SHOW HOW THIS ARISES AND IS NOT JUST A BRIDGE GIRDER, WHICH IS COVERED IN OTHER STANDARDS

Annex E [informative] Expressions for linear elastic stresses in rectangular plates from large deflection theory

THIS ANNEX WAS POORLY DRAFTED IN THE PREVIOUS VERSION OF THE STANDARD. IT WILL BE REPLACED WITH USEFUL APPROXIMATE EQUATIONS WHERE POSSIBLE. OTHERWISE GNA GLOBAL ANALYSIS SHOULD BE USED.

E.1 General

Annex F [informative] Expressions for reference resistance design of rectangular plates

F.1 General

(1) Direct design of unstiffened plate assemblies may be undertaken using the following treatments.

Dimensional limitations

Base support assumed

Load cases covered

Assumptions concerning plate junctions

Further material to be added here

F.2 Unstiffened plates in the form of a square pressurised box

Direct description of the required resistance of an unstiffened plate assembly to meet fully plastic design of the plate assembly

F.2 Unstiffened plates in the form of a rectangular pressurised box

Direct description of the required resistance of an unstiffened plate assembly to meet fully plastic design of the plate assembly