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Eurocode 3 : Design of steel structures

Part 3.1 : Towers, masts and chimneys – Towers and masts

Calcul des structures en acier

Partie 3.1 :

Tours, mâts et cheminées –
Pylônes et mats haubannés

Bemessung und Konstruktion von Stahlbauten

Teil 3.1 :

Türme, Maste und Schornsteine –
Türme und Maste

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Contents

Foreword

This document (prEN 1993-3:2004 "Towers, masts and chimneys") has been prepared on behalf of Technical Committee CEN/TC 250 "Structural Eurocodes", the secretariat of which is held be BSI.

This document is currently submitted to the Formal Vote.

This document will supersede ENV 1993-3.

Background of the Eurocode programme

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

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

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

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

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

EN 1990   Eurocode: Basis of structural design

EN 1991   Eurocode 1:   Actions on structures

EN 1992   Eurocode 2:   Design of concrete structures

EN 1993   Eurocode 3:   Design of steel structures

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

EN 1995   Eurocode 5:   Design of timber structures

EN 1996   Eurocode 6:   Design of masonry structures

EN 1997   Eurocode 7:   Geotechnical design

EN 1998   Eurocode 8:   Design of structures for earthquake resistance

EN 1999   Eurocode 9:   Design of aluminium structures

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

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

Status and field of application of Eurocodes

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

– as a means to prove compliance of building and civil engineering works with the essential requirements of Council Directive 89/106/EEC, particularly Essential Requirement N°1 - Mechanical resistance and stability - and Essential Requirement N°2 - Safety in case of fire;

– as a basis for specifying contracts for construction works and related engineering services;

– as a framework for drawing up harmonised technical specifications for construction products (ENs and ETAs)

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

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

National Standards implementing Eurocodes

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

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

– values for partial factors and/or classes where alternatives are given in the Eurocode,

– values to be used where a symbol only is given in the Eurocode,

– geographical and climatic data specific to the Member State, e.g. snow map,

– the procedure to be used where alternative procedures are given in the Eurocode,

– references to non-contradictory complementary information to assist the user to apply the Eurocode.

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

[3] According to Art. 12 of the CPD the interpretative documents should :
a) give concrete form to the essential requirements by harmonising the terminology and the technical bases and indicating classes or levels for each requirement where necessary ;
b) indicate methods of correlating these classes or levels of requirement with the technical specifications, e.g. methods of calculation and of proof, technical rules for project design, etc. ;
c) serve as a reference for the establishment of harmonised standards and guidelines for European technical approvals.
The Eurocodes, *de facto*, play a similar role in the field of the ER 1 and a part of ER 2.

Links between Eurocodes and product harmonised technical specifications (ENs and ETAs)

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

Additional information specific to EN 1993-3-1 and EN 1993-3-2

EN 1993-3 is the third part of six parts of EN 1993 - Design of Steel Structures - and describes the principles and application rules for the safety and serviceability and durability of steel structures for towers and masts and chimneys. Towers and masts are dealt with in Part 3-1; chimneys are treated in Part 3-2.

EN 1993-3 gives design rules in supplement to the generic rules in EN 1993-1.

EN 1993-3 is intended to be used with Eurocodes EN 1990 - Basis of design, EN 1991 - Actions on structures and the parts 1 of EN 1992 to EN 1998 when steel structures or steel components for towers and masts and chimneys are referred to.

Matters that are already covered in those documents are not repeated.

EN 1993-3 is intended for use by

– committees drafting design related product, testing and execution standards;
– clients (e.g. for the formulation of their specific requirements);
– designers and constructors;
– relevant authorities.

Numerical values for partial factors and other reliability parameters in EN 1993-3 are recommended as basic values that provide an acceptable level of reliability. They have been selected assuming that an appropriate level of workmanship and quality management applies.

Annex B of EN 1993-3-1 has been prepared to supplement the provisions of EN 1991-1-4 in respect of wind actions on lattice towers and guyed masts or guyed chimneys.

As far as overhead line towers are concerned all matters related to wind and ice loading, loading combinations, safety matters and special requirements (such as for conductors, insulators, clearance, etc.) are covered by the CENELEC Code EN 5034-1, that can be referred to for the design of such structures.

The strength requirements for steel members given in this Part may be considered as 'deemed to satisfy', rules to meet the requirements of EN 5034-1 for overhead line towers, and may be used as alternative criteria to the rules given in that Standard.
Part 3.2 has been prepared in collaboration with Technical Committee CEN / TC 297 : Free standing chimneys.

Provisions have been included to allow for the possible use of a different partial factor for resistance in the case of those structures or elements the design of which has been the subject of an agreed type testing programme.

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

National Annex for EN 1993-3-1

This standard gives alternative procedures, values and recommendations for classes with notes indicating where national choices may have to be made. Therefore the National Standard implementing EN 1993-3-1 should have a National Annex containing all Nationally Determined Parameters to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

National choice is allowed in EN 1993-3-1 through paragraphs:

- 2.1.1(3)
- 2.3.1(1)
- 2.3.2(1)
- 2.3.6(2)
- 2.3.7(1)
- 2.3.7(4)
- 2.5(1)
- 2.6(1)
- 4.2(1)
- 5.1(6)
- 5.2.4(1)
- 6.1(1)
- 6.4.1(1)
- 6.4.2(2)
- 6.5.1(1)
- 7.1(1)
- 9.5(1)
- A.1(1)
- A.2(1) (2 times)
- B.1.1(1)
- B.2.1.1(1)
- B.2.3(2)
- B.2.3(3)
- B.3.3.2.1(1)
- B.3.3.2.8(4)
- B.3.4(1)
- B.3.4(2)
- B.4.3.2.8.1(4)
- C.2(1)
- C.6.(1)
- D.1.1(1)
- D.1.2(2)
- D.3(6) (2 times)
- D.4.1(1)
- D.4.2(3)
- D.4.3(1)

- D.4.4(1)
- F.4.2.1(1)
- F.4.2.2(2)
- G.1(3)
- H.2(7)

1 General

1.1 Scope

1.1.1 Scope of Eurocode 3

See 1.1.1 of EN 1993-1-1.

1.1.2 Scope of Part 3.1 of Eurocode 3

(1)   This Part 3.1 of EN 1993 applies to the structural design of lattice towers and guyed masts and to the structural design of this type of structures supporting prismatic, cylindrical or other bluff elements. Provisions for the shafts of self-supporting and guyed cylindrical towers and chimneys are given in Part 3.2 of EN 1993. Provisions for the guys of guyed structures, including guyed chimneys, are given in EN 1993-1-11 and supplemented in this Part.

(2)   The provisions in this Part of EN 1993 supplement those given in Part 1.

(3)   Where the applicability of a provision is limited, for practical reasons or due to simplifications, its use is explained and the limits of applicability are stated.

(4)   This Part does not cover the design of polygonal and circular lighting columns, which is covered in EN 40. Lattice polygonal towers are not covered in this Part. Polygonal plated columns (monopoles) may be designed using this Part for their loading. Information on the strength of such columns may be obtained from EN 40.

(5)   This Part does not cover special provisions for seismic design, which are given in EN 1998-3.

(6)   Special measures that might be necessary to limit the consequences of accidents are not covered in this Part. For resistance to fire, reference should be made to EN 1993-1-2.

(7)   For the execution of steel towers and masts, reference should be made to EN 1090.

> **NOTE:**   Execution is covered to the extent that is necessary to indicate the quality of the construction materials and products that should be used and the standard of workmanship on site needed to comply with the assumptions of the design rules.

(8)   The following subjects are dealt with in EN 1993-3-1:

Section 1:   General

Section 2:   Basis of design

Section 3:   Materials

Section 4:   Durability

Section 5:   Structural analysis

Section 6:   Ultimate limit states

Section 7:   Serviceability limit states

Section 8:   Design assisted by testing

Section 9:   Fatigue

1.2 Normative references

The following normative documents contain provisions which, through reference in this text, constitute provisions of this European Standard. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. However, parties to agreements based on this European Standard are

encouraged to investigate the possibility of applying the most recent editions of the normative documents indicated below. For undated references, the latest edition of the normative document referred to applies.

EN 40   *Lighting columns*

EN 365

EN 795

EN 1090   *Execution of steel structures*

EN ISO 1461

EN ISO 14713

ISO 12494   *Atmospheric icing of structures*

EN ISO 12944 *Corrosion protection*

1.3 Assumptions

(1)   See 1.3 of EN 1993-1-1.

1.4 Distinction between principles and application rules

(1)   See 1.4 of EN 1993-1-1.

1.5 Terms and definitions

(1)   The terms and definitions that are defined in EN 1990 clause 1.5 for common use in the Structural Eurocodes apply to this Part 3.1 of EN 1993.

(2)   Supplementary to Part 1 of EN 1993, for the purposes of this Part 3.1, the following definition apply:

1.5.1
global analysis
the determination of a consistent set of internal forces and moments in a structure, that are in equilibrium with a particular set of actions on the structure.

1.5.2
tower
a self-supporting cantilevered steel lattice structure of triangular, square or rectangular plan form, or circular and polygonal monopoles.

1.5.3
guyed mast
a steel lattice structure of triangular, square or rectangular plan form, or a cylindrical steel structure, stabilized at discrete intervals in its height by guys that are anchored to the ground or to a permanent structure.

1.5.4
shaft
the vertical steel structure of a mast.

1.5.5
leg members
steel members forming the main load-bearing components of the structure.

1.5.6
primary bracing members
members other than legs, carrying forces due to the loads imposed on the structure.

1.5.7

secondary bracing members

members used to reduce the buckling lengths of other members.

1.5.8

schifflerized angles

modified 90° equal-leg hot rolled angles, each leg of which has been bent to incorporate a 15° bend such that there is an angle of 30° between the outer part of each leg and the axis of symmetry (see Figure 1.1).

1.5.9

wind drag

the resistance to the flow of wind offered by the elements of a tower or guyed mast and any ancillary items that it supports, given by the product of the drag coefficient and a reference projected area, including ice where relevant.

1.5.10

linear ancillary item

any non-structural components that extend over several panels, such as waveguides, feeders, ladders and pipework.

1.5.11

discrete ancillary item

any non-structural component that is concentrated within a few panels, such as dish reflectors, aerials, lighting, platforms, handrails, insulators and other items.

1.5.12

projected area

the shadow area of the element considered, when projected on to an elevation parallel to the face of the structure normal to the wind direction considered, including ice where relevant. For wind blowing other than normal to one face of the structure, a reference face is used for the projected area. (See Annex A and Annex B.)

1.5.13

panel (of a tower or mast)

any convenient portion of a tower or mast that is subdivided vertically for the purpose of determining projected areas and wind drag. Panels are typically, but not necessarily, taken between intersections of legs and primary bracings.

1.5.14

section (of a tower or mast)

any convenient portion of a tower or mast comprising several panels that are nearly or exactly similar, used for the purpose of determining wind drag.

1.5.15

guy

a tension-only member, connected at each end to terminations to form a guy assembly that provides horizontal support to the mast at discrete levels. The lower end of the guy assembly is anchored to the ground or on a structure and generally incorporates a means of adjusting the tension in the guy.

> **NOTE 1:** Although the terms "stay" and "guy" are generally interchangeable, the word "guy" has been used throughout this document.

> **NOTE 2:** Specific definitions of guys, their make-up and fittings, are provided in Annex D.

1.5.16

damper

a device that increases the structural damping or limits the response of a structure or of a guy.

1.6   Symbols

(1)   In addition to those given in EN 1993-1-1, the following main symbols are used:

Latin upper case letters

D_b	diameter of the circle through the centre of the bolt hole
D_i	diameter of the leg member
G	gust response factor
M	bending moment
N	tension force, number of cycles
N_i	number of cycles
N_b	axial force
T	design life of the structure in years

Latin lower case letters

b	width of a leg of an angle
$c_e(z)$	exposure factor
$c_s c_d$	structural factor
e	eccentricities
h	width of a leg of an angle
k_p	prying effect factor
k_σ	buckling coefficient
m	slope of the S-N curve
n	number of bolts
r_1	radius of the convex part of the bearing
r_2	radius of the concave part of the bearing
t	thickness

Greek upper case letters

ϕ	is the inclination of the mast axis at its base
$\Delta\sigma_E$	stress range

Greek lower case letters

β_A	factor for effective area
γ_M	partial factor
δ_s	logarithmic decrement of structural damping
ε	coefficient depending on f_y
$\overline{\lambda}$	non-dimensional slenderness parameter, equivalence factor
$\overline{\lambda}_p$	non-dimensional slenderness for plate buckling
$\overline{\lambda}_{p,1}$	non-dimensional slenderness parameter for plate buckling of leg 1 of angle
$\overline{\lambda}_{p,2}$	non-dimensional slenderness parameter for plate buckling of leg 2 of angle
ρ	reduction factor
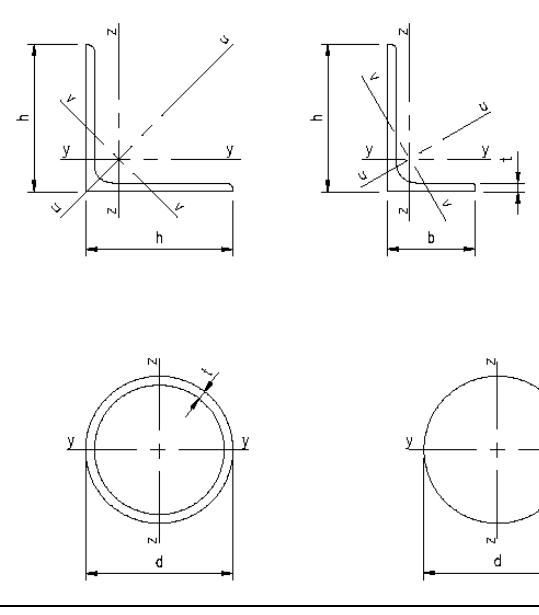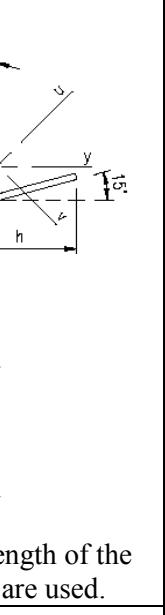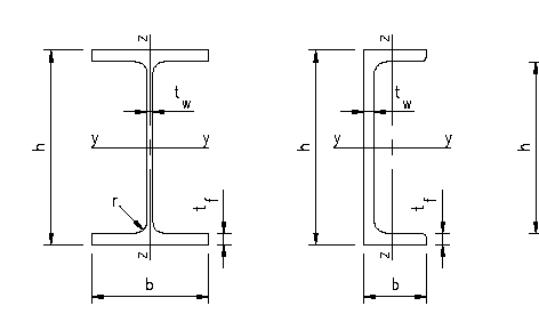
(2)   Further symbols are defined where they first occur.

1.7   Convention for cross section axes

(1)   The convention for axes of angle sections adopted in this Part of EN 1993 is as shown in Figure 1.1.

NOTE: This avoids the confusion inherent in adopting different conventions for hot rolled angles and cold formed angles.

(2)   For built-up members the convention for axes is that of Figure 6.9 of EN 1993-1-1.

schifflerized angle

NOTE: h is taken as the longest length of the individual angle, if unequal angles are used.

Figure 1.1 Dimensions and axes of sections

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

(1)   The design of steel towers and guyed masts should be in accordance with the general rules given in EN 1990.

(2)   The provisions for steel structures given in EN 1993-1-1 should also be applied.

(3)   In addition, guyed masts of high reliability (as defined in 2.1.2) should be designed to withstand the rupture of one guy without collapsing.

> **NOTE:** The National Annex may give information on guy rupture. It is recommended to use the guidance given in Annex E.

2.1.2 Reliability management

(1)   Different levels of reliability may be adopted for the ultimate limit state verifications of towers and masts, depending on the possible economic and social consequences of their collapse.

> **NOTE:** For the definition of different levels of reliability see Annex A.

2.2 Principles of limit state design

(1)   See 2.2 of EN 1993-1-1.

2.3 Actions and environmental influences

2.3.1 Wind actions

(1)   Wind actions should be taken from EN 1991-1-4.

> **NOTE:** The National Annex may give information on how EN 1991-1-4 could be supplemented for masts and towers. The use of the additional rules given in Annex B is recommended.

2.3.2 Ice loads

(1)   Actions from ice should be considered both by their gravity effects and their effect on wind actions.

> **NOTE:** The National Annex may give information on ice loading, the appropriate ice thicknesses, densities and distributions and appropriate combinations, and combination factors for towers and masts. The use of Annex C is recommended.

2.3.3 Thermal actions

(1)   Thermal actions should be determined from EN 1991-1-5 for environmental temperatures.

2.3.4 Selfweight

(1)   Selfweight should be determined in accordance with EN 1991-1-1.

(2)   Selfweight of guys should be determined in accordance with EN 1993-1-11.

2.3.5   Initial guy tensions

(1)   The initial guy tensions should be considered as effective permanent forces, existing in the guys in the absence of meteorological actions, see EN 1993-1-11.

(2)   Adjustment for initial guy tensions should be provided. If not, due allowance should be taken in design for the range of initial tensions that might arise, see EN 1993-1-11.

2.3.6   Imposed loads

(1)   Members that are within [30°] to the horizontal should be designed to carry the weight of a workman which for this purpose may be taken as a concentrated vertical load of 1kN.

(2)   Imposed loads on platforms and railing should be taken into account.

> **NOTE 1:** The National Annex may give information on imposed loads on platforms and railings. The following characteristic imposed loads are recommended:
>
> –   Imposed loads on platforms: **2,0 kN/m^2**   ... (2.1a)
> –   Horizontal loads on railings: **0,5 kN/m**   ... (2.1b)

> **NOTE 2:** These loads may be assumed to act in the absence of other climatic loads.

2.3.7   Other actions

(1)   For accidental and collision actions see EN 1991-1-7.

> **NOTE:** The National Annex may give information on the choice of accidental actions.

(2)   Actions during execution should be considered taking due account of the construction scheme. The appropriate load combinations and reduction factors may be obtained from EN 1991-1-6.

> **NOTE:** The limited time for transient design situations may be considered.

(3)   Where considered necessary, actions from settlement of foundations should be assessed. Special considerations may be required for lattice towers founded on individual leg foundations and for differential settlement between the mast base and any guy foundations.

(4)   Actions arising from the fitting and anchoring of safety access equipment may be determined with reference to EN 795. Where the proposed safe method of working requires the use of Work Positioning Systems or mobile fall arrest systems points of attachment should be adequate, see EN 365.

> **NOTE:** The National Annex may give further information.

2.3.8   Distribution of actions

(1)   The loads along the member length including wind or dead loading on other members framing into the member should be considered.

2.4   Ultimate limit state verifications

(1)   For design values of actions and combination factors see EN 1990.

> **NOTE:** For partial factors for actions in the ultimate limit state see Annex A.

(2)   The partial factors for gravity loads and initial tensions in guys should be taken as specified in EN 1993-1-11.

2.5   Design assisted by testing

(1)   The general requirements specified in EN 1990 should be satisfied, in association with the specific requirements given in Section 8 of this Part 3.1 of EN 1993.

> **NOTE:** The National Annex may give further information for structures or elements that are subject to an agreed full-scale testing programme, see 5.1.1.3.

2.6   Durability

(1)   Durability should be satisfied by complying with the fatigue assessment (see section 9) and appropriate corrosion protection (see section 4).

> **NOTE:** The National Annex may give information on the design service life of the structure. A service life of 30 years is recommended.

3   Materials

3.1   Structural steel

(1)   For requirements and properties for structural steel, see EN 1993-1-1 and EN 1993-1-3.

3.2   Connections

(1)   For requirements and properties for bolts and welding consumables, see EN 1993-1-8.

3.3   Guys and fittings

(1)   For requirements and properties of ropes, strands, wires and fittings see EN 1993-1-11.

NOTE: See also Annex D

3.4   Toughness

(1)   For toughness requirements see EN 1993-1-10.

4   Durability

4.1   Allowance for corrosion

(1)   Suitable corrosion protection, appropriate to the location of the structure, its design life and maintenance regime, should be provided.

NOTE 1: The National Annex may give further information.

NOTE 2: See also:

–   EN ISO 1461 for galvanising,

–   EN ISO 14713 for metal spraying and

–   ISO/EN 12944 for corrosion protection by painting.

4.2   Guys

(1)   For guidance on the corrosion protection of guys see EN 1993-1-11.

NOTE: The National Annex may give further informations. The following measures are recommended:

Dependent on the environmental conditions guy ropes made from galvanized steel wires should be given a further layer of protection, such as grease or paint. Care should be taken to ensure that this protective layer is compatible with the lubricant used in the manufacture of the guy ropes.

As an alternate means of protection galvanised steel ropes of diameter up to 20mm may be protected by polypropylene impregnation in which case they do not need further protection unless the sheath is damaged during erection and use. Care needs to be taken in designing the terminations to ensure adequate corrosion protection. Non-impregnated sheathed ropes should not be used because of the risk of corrosion taking place undetected.

5 Structural analysis

5.1 Modelling for determining action effects

(1)   The internal forces and moments should be determined using elastic global analysis.

(2)   For elastic global analysis see EN 1993-1-1.

(3)   Gross cross-sectional properties may be used in the analysis.

(4)   Account should be taken of the deformation characteristics of the foundations in the design of the structure.

(5)   If deformations have a significant effect (for example towers with large head-loads) second order theory should be used, see EN 1993-1-1.

> **NOTE 1:** Lattice towers may initially be analysed using the initial geometry (first order theory).

> **NOTE 2:** Masts and guyed chimneys should be analysed taking into account the effect of deformations on the equilibrium conditions (second order theory).

> **NOTE 3:** Overall buckling of masts should be properly accounted for in the analysis of symmetric masts (see B.4.3.2.6).

(6)   The global analysis of a mast or guyed chimneys should take into account the non-linear behaviour of the guys, see EN 1993-1-11.

> **NOTE:** The National Annex may give further information.

5.2 Modelling of connections

5.2.1 Basis

(1)   The behaviour of the connections should be considered in the global and local analysis of the structure.

> **NOTE:** The procedure for the analysis of connections is given in EN 1993-1-8.

5.2.2 Fully triangulated structures (Simple framing)

(1)   In simple framing the connections between the members may be assumed not to develop moments. In the global analysis, members may be assumed to be effectively pin connected.

(2)   The connections should satisfy the requirements for nominally pinned connections, either:

– as given in 5.2.2.2 of EN 1993-1-8; or
– as given in 5.2.3.2 of EN 1993-1-8.

5.2.3 Non-triangulated structures (Continuous framing)

(1)   Elastic analysis should be based on the assumption of full continuity, with rigid connections which satisfy the requirements given in 5.2.2.3 of EN 1993-1-8.

5.2.4 Triangulated structures where continuity is taken into account (continuous or semi-continuous framing)

(1)   Elastic analysis should be based on reliably predicted design moment-rotation or force-displacement characteristics for the connections used.

> **NOTE:** The National Annex may give further information.

6 Ultimate limit states

6.1 General

(1) The following partial factors γ_M apply:

– resistance of member to yielding: γ_{M0}

– resistance of member buckling: γ_{M1}

– resistance of net section at bolt holes: γ_{Mb}

– resistance of connections: See Section 6.4

– resistance of guys and their terminations: γ_{Mg}, see EN 1993-1-11

– resistance of insulating material: γ_{Mi}

> **NOTE 1:** The National Annex may give information on partial factors γ_M. The following values are recommended:
>
> $\gamma_{M0} = 1,00$
>
> $\gamma_{M1} = 1,10$
>
> $\gamma_{Mb} = 1,25$
>
> $\gamma_{Mg} = 2,00$
>
> $\gamma_{Mi} = 2,50$

> **NOTE 2:** The factor γ_{Mg} applies to the guy and its socket (or other termination). The associated steel pins, linkages and plates are designed for compatibility with the guy and its socket and may require an enhanced value of γ_{Mg}. For details see EN 1993-1-11.

> **NOTE 3:** For structures or elements that are to be type tested, or where similar configurations have previously been type tested the partial factors, γ_M, may be reduced, subject to the outcome of the testing programme.

6.2 Resistance of cross sections

6.2.1 Classification of cross sections

(1) For towers and masts, classification of cross-sections as given in 5.5.2 of EN 1993-1-1 should be used.

> **NOTE**: The maximum width to thickness ratio c/t for angles defined in table 5.2 of EN 1993-1-1 may be determined with the ratio (h-2t)/t instead of h/t.

6.2.2 Members in lattice towers and masts

(1) For angles connected by one leg, special provisions are given in 3.9.3 of EN 1993-1-8 (if bolted) or 4.1.3 (if welded).

6.2.3 Guys and fittings

(1) For the strength of guys and fittings see EN 1993-1-11 and Annex D.

6.3 Resistance of members

6.3.1 Compression members

(1) Compression members in lattice towers and masts should be designed according to EN 1993-1-1

taking account of eccentricities.

> **NOTE 1:** For buckling of components of masts and towers see Annex G.

> **NOTE 2:** Information about buckling length and slenderness of members in towers and masts is given in Annex H.

(2)   The effective cross section properties of members should be calculated according to 4.3 of EN 1993-1-5.

> **NOTE 1:** For angles the reduction factor ρ may be determined with the slenderness $\overline{\lambda}_p$ taking into account the appropriate width $\overline{b}$ of the compression leg as follows :

a) for equal leg angles:

$$\overline{\lambda}_p = \frac{\overline{b}/t}{28,4\,\varepsilon\,\sqrt{k_\sigma}} = \frac{(h-2t)/t}{28,4\,\varepsilon\,\sqrt{k_\sigma}}$$

b) for unequal leg angles:

$$\overline{\lambda}_{p,1} = \frac{\overline{b}/t}{28,4\,\varepsilon\,\sqrt{k_\sigma}} = \frac{(h-2t)/t}{28,4\,\varepsilon\,\sqrt{k_\sigma}} \text{ and}$$

$$\overline{\lambda}_{p,2} = \frac{\overline{b}/t}{28,4\,\varepsilon\,\sqrt{k_\sigma}} = \frac{(b-2t)/t}{28,4\,\varepsilon\,\sqrt{k_\sigma}}$$

> **NOTE 2:** In the case of angles connected by one leg, the reduction factor, ρ, only applies to the connected leg.

(3)   For cases of low slenderness, the torsional and/or flexural-torsional mode should also be checked, see 6.3.1.4 of EN 1993-1-1 and EN 1993-1-3.

> **NOTE:** For equal leg angles with $\overline{\lambda} < \dfrac{5h}{\lambda_1 t}$ local buckling will govern and the non dimensional slenderness may be estimated with: $\overline{\lambda} = \dfrac{h}{t}\sqrt{\dfrac{2,6 f_y \beta_A}{E}}$.

(4)   For cold formed thin gauge members see EN 1993-1-3.

6.4   Connections

6.4.1   General

(1)   For connections see EN 1993-1-8.

> **NOTE:** The partial factors for connections in masts and towers may be given in the National Annex. The numerical values given in Table 2.1 of EN 1993-1-8 are recommended.

6.4.2   Tension bolts in end plates (flanged connections)

(1)   Where there is a possibility of tension across the flange connection preloaded bolts should be used.

(2)   The minimum bolt diameter should be 12mm.

> **NOTE:** The National Annex may give further information on flange connections of circular hollow

sections and cylindrical shells. For circular hollow sections the following simplified method for members in tension without bending is recommended, see figure 6.1.

In determining the flange thickness the following is relevant:

a) the shear resistance of the flange along the perimeter of the connected circular leg section;

b) the resistance to combined bending and shear of the flange along the circle through the bolt holes. The bending moment (M) may be taken as:

$M = N (D_b – D_i)/2$

where: N   is the tension force in the leg member

D_b is the diameter of the circle through the centre of the bolt holes

D_i is the diameter of the leg member

combined bending and shear to be checked

shear to be checked
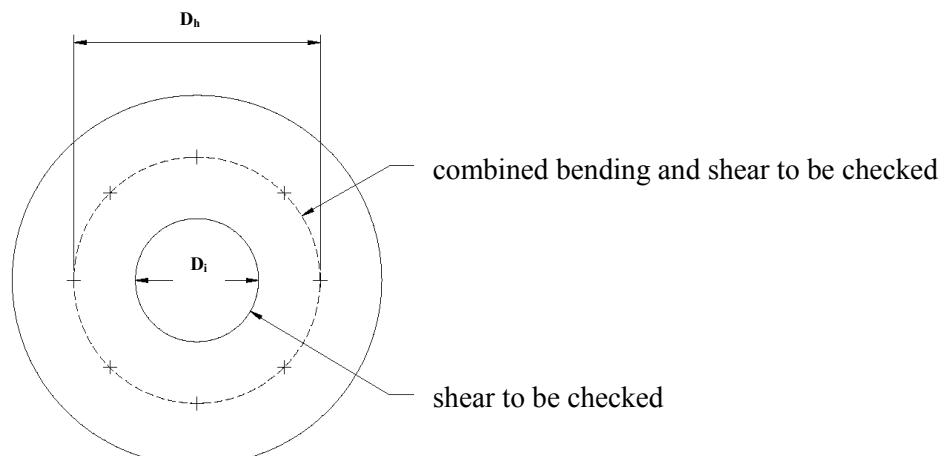
Figure 6.1 Bolted flanged connections

In determining the forces in the bolts, the axial force N_b

$$N_b = \frac{N k_p}{n}$$

where: n   is the number of bolts

k_p is a prying effect factor taken as

k_p =   1,2 for pre-loaded bolts

=   1,8 for non-preloaded bolts

6.4.3   Anchor bolts

(1)   Where fatigue needs to be considered anchor bolts should be preloaded. In such cases appropriate steel materials should be used, see EN 1993-1-8.

NOTE: For the choice of the preload see also rules for prying force eccentricity, stress levels etc. in EN 1993-1-8.

6.4.4   Welded connections

(1)   See EN 1993-1-8.

NOTE: For execution see EN 1090.

6.5   Special connections for masts

6.5.1   Mast base joint

(1)   The design bearing stress on the spherical pinned connection should be based on the design rules for rocker bearings, see EN 1337-6.

NOTE: The National Annex may give information on eccentricities and limit values for the Hertz pressure.

To verify that the area of the compression zone is within the boundaries of the bearing parts taking due account of the true rotation angle of the mast base section (see Figure 6.2) and to determine the bending moments caused by the resulting eccentricities for designing the bearing and the bottom section of the mast the following rules for determining eccentricities are recommended:

If the mast base rests on a spherical bearing the point of contact should be assumed to move in the direction of any inclination of the mast axis by rolling over the bearing surface.

The eccentricities e_u and e_o (see Figure 6.2) should be determined as follows:

$$e_u = r_1 \times \sin\psi_1 \qquad\qquad ...(6.12a)$$

$$e_o = r_2 (\sin \psi_1 - \sin \phi) \qquad\qquad ...(6.12b)$$

where: r_1 is the radius of the convex part of the bearing;

   r_2 is the radius of the concave part of the bearing;

and $r_2 > r_1$

ϕ is the inclination of the mast axis at its base.

with: $\psi_1 = \dfrac{r_2\phi}{r_2 - r_1}$ \qquad\qquad ...(6.13a)

   $\psi_2 = \psi_1 - \phi$ \qquad\qquad ...(6.13b)

If r_2 is infinite, that is a flat surface, then e_o should be taken as $e_o = r_1 \phi \cos\phi$.

1 mast axis

*2 area of compression zone*
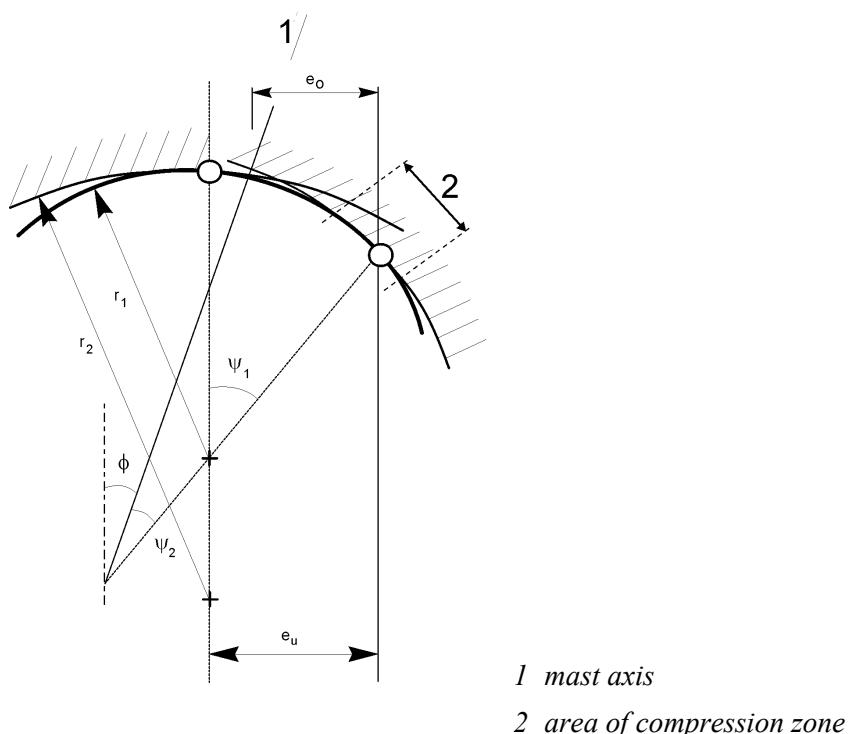
Figure 6.2 Eccentricities due to the inclination of the mast base

(2)   The system for suppressing twisting of a pinned mast base joint should be designed to permit rotation of the mast base section around the horizontal axes.

(3)   For a fixed mast base possible settlements of the shaft foundation and of the guy foundations should be considered in the mast design.

6.5.2   Guy connections

(1)   All connections of the guys to the mast or to guy foundations should allow the guy to rotate freely in both vertical and horizontal directions, see EN 1993-1-11.

Account should be taken in the design and detailing connections of the tendency for guy constructions to twist under tensile loading.

> **NOTE:** Generally for connections with pins the freedom for horizontal rotations can be obtained by a "spherical" form of the hole in the centre plate for the pin. Spherical bearings may be used in exceptional circumstances.

(2)   All pins should be adequately secured against lateral movement by the use, for example, of a nut combined with a split pin.

(3)   The guy attachment plate in the mast and the steel anchor plate projecting from the guy foundation should both be designed for the lateral force from the guy due to the wind loading component normal to the plane of the guy.

(4)   Wherever practicable welded connections should be detailed to enable visual and non-destructive inspections to be undertaken in service.

7 Serviceability limit states

7.1 Basis

(1)   The following serviceability limit states may be relevant for design:

– deflections or rotations that adversely affect the effective use of the structure, including the proper functioning of aerials or services;

– vibration, oscillation or sway that causes loss of transmitted signals;

– deformations, deflections, vibration, oscillation or sway that causes damage to non-structural elements.

> **NOTE:** The National Annex may give information on limits and associated γ_M-values. The value $\gamma_M = 1,0$ is recommended.

7.2 Deflections and rotations

7.2.1 Requirements

(1)   The maximum deflections and rotations should be determined using the combination of characteristic actions on the structure and its ancillaries.

(2)   The deflections and rotations for masts and guyed chimneys should be calculated making due allowance for any second order effects, see EN 1993-1-1, and any dynamic effects.

7.2.2 Definition of limiting values

(1)   Limiting values should be specified together with the load case considered.

> **NOTE:** For guyed masts this will be the maximum deflection under patch loading see Annex B.

(2)   For broadcasting and floodlighting structures, the limiting values to be considered should be taken as those for horizontal displacement and rotation at the top of the structure. For directional antennae the limiting values should be taken at the point of the attachment of the directional antenna.

7.3 Vibrations

(1)   Towers and masts should be examined for:

– gust induced vibrations (causing vibrations in the direction of the wind);

– vortex induced vibrations for towers or masts containing prismatic cylindrical or bluff elements or shrouds (causing vibrations perpendicular to the direction of the wind);

– galloping instability (causing vibrations of the guys);

– rain-wind induced vibrations.

> **NOTE 1:** For dynamic effects see EN 1991-1-4 and Annex B.

> **NOTE 2:** Vibrations can cause rapid development of fatigue damage, see section 9.

(2)   If lattice towers and masts or guyed chimneys are predicted to be subject to wind vibrations, unless other measures are taken to reduce these in the design, provisions should be made for the installation of damping devices if found necessary in the light of experience.

> **NOTE:** See Annex B of EN 1993-3-2.

8   Design assisted by testing

(1)   The provisions for design assisted by testing given in EN 1990 should be followed.

(2)   Where the values given in EN 1991-1-4 are considered inappropriate for lattice towers and masts consisting of, supporting or containing cylindrical elements, testing may be undertaken to determine the logarithmic decrement of structural damping, δ_s.

> **NOTE:** Guidance for the determination of δ_s is given in Annex C to EN 1993-3-2.

(3)   Higher modes than the fundamental might be significant, particularly for guyed masts, so due account of this should be taken in determining the appropriate logarithmic decrement of structural damping.

(4)   Account should be taken of the fact that the frequencies of vibration vary according to the loading condition considered for instance in still air, under wind, or under ice loading.

9   Fatigue

9.1   General

(1)   For fatigue verifications the provisions of EN 1993-1-9 should be applied.

(2)   Consideration should be given to the effects on fatigue resistance of the possible existence of secondary moments in lattice towers and masts that are not already allowed for.

9.2   Fatigue loading

9.2.1   In-line vibrations

(1)   Fatigue loading of lattice towers due to in-line vibrations (without cross-wind vibrations) induced by gusty wind need not be determined.

> **NOTE:** For guyed masts provided that the detail category of the structural details are greater than 71 N/mm^2, the fatigue life of structures subject to in-line vibrations only (without cross-wind vibrations) induced by gusty wind may be assumed to be greater than 50 years.

(2)   In all other cases due account should be taken of the details adopted, and fatigue verification undertaken.

> **NOTE:** For the fatigue verification due to in-line vibrations see EN 1991-1-4. The following simplified method may be used:
>
> a)   The fatigue stress history due to wind gusts is evaluated by determining the annual durations of different mean wind speeds from different directions from meteorological records for the site. The fluctuations about the mean values may then be assumed to have a statistically normal distribution with a standard deviation in stress corresponding to G/4 times the stress due to the mean wind speed. The appropriate gust response factor G is defined as
>
> $$G = c_e(z)\, c_s c_d - 1$$
>
> where
>
> $c_e(z)$ is the exposure factor, see EN 1991-4
>
> $c_s c_d$ is the structural factor, see EN 1991-4
>
> derived in accordance with Annex B.
>
> b)   The stress range, $\Delta\sigma_{Si}$, may be assumed to be 1,1 times the difference between the stress arising from that incorporating the gust response factor G and that due to the ten minute mean wind speed. An equivalent number of cycles N_i may then be obtained from:
>
> $$N_i \;=\; 10^5\, T/50 \qquad\qquad\qquad \text{... (9.1)}$$
>
> where: T is the design life of the structure in years.

9.2.2   Cross-wind vibrations

(1)   The fatigue loading of towers and guyed masts consisting of, supporting or containing prismatic, cylindrical or other bluff elements should be determined from the maximum amplitude for the relevant vibration mode and the number of stress cycles N.

> **NOTE:** For the fatigue actions see EN 1991-1-4, Annex E.

9.2.3 Individual member response

(1)   Slender individual members of structures should be assessed for cross-wind excitation.

> **NOTE:** For the fatigue actions see EN 1991-1-4, Annex E. The limitations on slenderness given in Annex H H.2(1) and H.3.1(3) will generally be sufficient to prevent such excitation. An increase of damping (friction, additional dampers) is a practical means of suppressing such vibrations if they occur in practice.

9.3   Fatigue resistance

(1)   Reference should be made to EN 1993-1-9 which includes resistances of details typical for towers, chimneys and guyed masts.

9.4   Safety assessment

(1)   The safety assessment for fatigue should be carried out in accordance with 8(2) of EN 1993-1-9, using:

$$\Delta\sigma_{E2} = \lambda\Delta\sigma_E \qquad\qquad ... (9.2)$$

where:

λ is the equivalence factor to transfer $\Delta\sigma_s$ to $N_c = 2 \times 10^6$ cycles;

$\Delta\sigma_E$ is the stress range associated to N cycles (see 9.2) allowing for stress concentration factors where appropriate.

(2)   The equivalence factor λ may be determined from:

$$\lambda = \left(\frac{N}{2\times10^6}\right)^{\frac{1}{m}} \qquad\qquad ... (9.3)$$

where: m   is the slope of the S-N curve.

9.5   Partial factors for fatigue

(1)   The partial factors for fatigue should be taken as specified in 3(6) and (7) and 6.2(1) of EN 1993-1-9.

> **NOTE:** The National Annex may give numerical values for γ_{Ff} and γ_{Mf}. For γ_{Ff} the value $\gamma_{Ff} = 1,00$ is recommended. For γ_{Mf} values see Table 3.1 in EN 1993-1-9.

9.6   Fatigue of guys

(1)   The fatigue performance of guys should be verified using the procedures given in EN 1993-1-11.

Annex A [normative] – Reliability differentiation and partial factors for actions

NOTE: As this Annex deals with reliability differentiation and partial factors for actions for masts and towers, it is expected that it will be transferred to Annex A to EN 1990 in a later stage.

A.1   Reliability differentiation for masts and towers

(1)   Reliability differentiation may be applied to masts and towers by the application of reliability classes.

NOTE: The National Annex may give relevant reliability classes related to the consequences of structural failure. The classes in Table A.1 are recommended.

Table A.1 Reliability differentiation for towers and masts

Reliability Class	
3	towers and masts erected in urban locations, or where their failure is likely to cause injury or loss of life; towers and masts used for vital telecommunication facilities; other major structures where the consequences of failure would be likely to be very high
2	all towers and masts that cannot be defined as class 1 or 3
1	towers and masts built on unmanned sites in open countryside; towers and masts, the failure of which would not be likely to cause injury to people

A.2   Partial factors for actions

(1)   Partial factors for actions should be dependant on the reliability class of the tower or mast.

NOTE 1: In the choice of partial factors for permanent actions γ_G and for variable actions γ_Q the dominance of wind actions for the design may be taken into account.

NOTE 2: The National Annex may give numerical values of γ_G and γ_Q. Where the reliability classes recommended in A.1 are used the numerical values in Table A.2 for γ_G and γ_Q are recommended.

Table A.2 Partial factors for permanent and variable actions

Type of Effect	Reliability Class, see NOTE to 2.1.2	Permanent Actions	Variable Actions (Qs)
unfavourable	3	1,2	1,6
	2	1,1	1,4
	1	1,0	1,2
favourable	All Classes	1,0	0,0
Accidental situations		1,0	1,0

NOTE 3: The National Annex may also give information on the use of dynamic response analysis for wind actions, see Annex B.

Annex B [informative] – Modelling of meteorological actions

> **NOTE:** As this Annex deals with supplementary rules for wind actions on lattice towers, guyed masts and guyed chimneys, and on their response, it is expected that it will be transferred to EN 1991-1-4 in a later stage.

B.1 General

B.1.1 Scope of this Annex

(1)   This Annex is a supplementation to prEN 1991-1-4 for wind loads and wind actions on towers and masts. It should be used for the determination of wind loads and wind actions on single members of arbitrary profiles while prEN 1991-1-4 can be used for the determination of the overall wind load for masts with straight legs either with rounded or rectangular profile. It can be used for the following wind actions:

– wind force, see B.2;

– response of lattice towers, see B.3; and

– response of guyed masts, see B.4.

> **NOTE:** This Annex refers to ISO 12494 for ice loading. The National Annex may give further information.

B.1.2 Symbols

(1)   In addition to those given in EN 1993-1-1 and EN 1991-1-4, the following main symbols have been used in this Annex:

g	peak factor
i	patch load pattern
K	factor
L	projected length, chord length
N	number
Q	parameter
S	load effect in a member (e.g. force, shear or bending moment)
T	torque
α	slope of guy to horizontal
β	parameter
η	shielding factor
θ	angle of wind incidence to the normal in plane; slope
τ	constant
ψ	angle of wind incidence to the longitudinal axis
ω	spacing ratio

(2)   In addition to those given in EN 1993-1-1 the following subscripts have been used in this Annex:

A	ancillary item
C	cantilever
c	circular-section members
e	effective
F	face
f	flat-sided members
G	guy
H	mast height
L	length
M	base mast or mast only
m	mast; mean
n	single frame

PL	patch load
p	patch
q	shear
S	structure
sup	super-critical
T	tower, total
W	in the direction of the wind
w	with wind
x	in the crosswind direction
Z	in the vertical direction
z	height z above ground level
θ	angle of wind incidence

B.2   Wind force

B.2.1   General

B.2.1.1 Outline

(1)   For the purposes of calculating the wind force, the structure should be divided into a series of sections, where a section comprises several identical or nearly identical panels, see Figure B.2.1. Projections of bracing members in faces parallel to the wind direction, and in plan and hip bracing, should be omitted in the determination of the projected area of the structure.

(2)   The structure should generally be divided into a sufficient number of sections to enable the wind loading to be adequately modelled for the global analysis.

(3)   The wind force acting on a section or component should be determined according to 5.3 (2) of EN 1991-1-4.

(4)   In determining the wind force under iced conditions, the projected areas of structural elements and ancillaries should be increased to take due account of the thickness of ice as relevant.

(5)   In applying the method given in this Annex, the maximum force within an angle of ± 30° to the nominal wind direction should be used to obtain the maximum loading in the wind direction.

> **NOTE:** The National Annex may give information of wind tunnel tests.

B.2.1.2 Method

(1)   The method given in B.2.1.3 should be used to determine the drag of square or equilateral triangular lattice structures.

> **NOTE 1:** The procedure given in B.2.7 only applies for either:
> a) as guidance for structures of rectangular cross section; or
> b) the assessment of existing structures for which the disposition of ancillaries and aerials is accurately known.

> **NOTE 2:** The procedure given in B.2.7 may provide lower values of drag than the method given in B.2.1.3 when K_A is taken as 1,0 in B.2.3 and B.2.4.

B.2.1.3 Total wind force coefficient
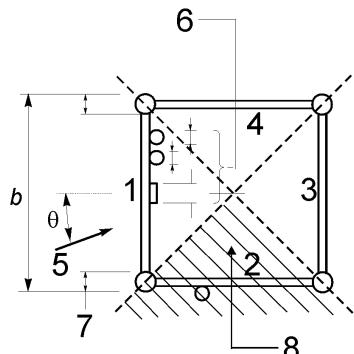
(1)   The total wind   force coefficient c_f in the direction of the wind over a section of the structure should be taken as:

$$\Sigma c_f = c_{f,S} + c_{f,A}$$
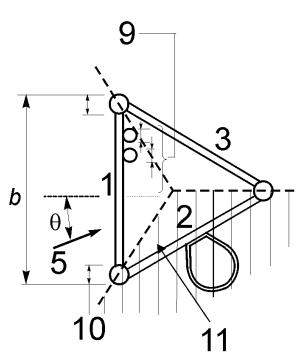
... (A.1)

where: $c_{f,S}$ is the wind force coefficient of the bare structure section, determined in accordance with B.2.2 using the solidity ratio, φ, appropriate to the bare structure; and

$c_{f,A}$ is the wind force coefficient of the ancillaries, determined in accordance with B.2.3 and B.2.4, as appropriate.

(2)   Where the projected areas of ancillaries on each face are within 10% of each other, then they may be treated as appropriate structural members and the total wind drag calculated in accordance with B.2.2.
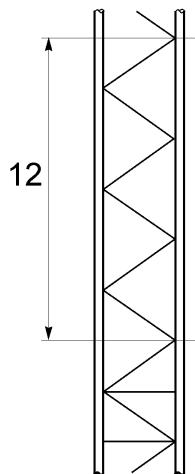

NOTE: Face 1 should be taken as windward such that $-45° \le \theta \le 45°$

(a) Plan on square structure
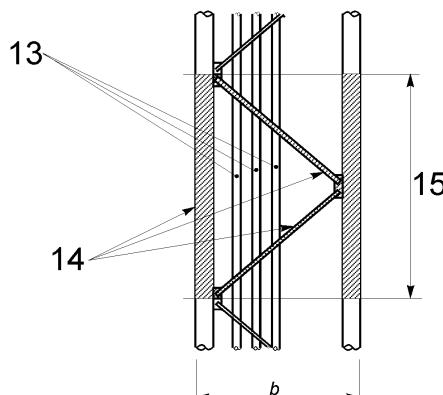
NOTE: Face 1 should be taken as windward such that $-60° \le \theta \le 60°$. External ladder should be treated as individual item.

(b) Plan on triangular structure

1   Face 1
2   Face 2
3   Face 3
4   Face 4
5   Wind
6   Ancillary components projected normal to face 1
7   Leg projected normal to face
8   Ancillary components in this area allocated to face 2
9   Ancillary components projected normal to face (inclusive of ladder rungs, hoops etc.)
10 Leg projected normal to face
11 Ancillary components in this area allocated to face 2
12 Mast section
13 Ancillary components of projected area A_A
14 Structural components of projected area A_S
15 Panel height (h)

(c) Mast section

(d) Structural panel

Figure B.2.1 Projected panel area used to calculate solidity ratio, φ

B.2.2   Wind force coefficient of structural components

B.2.2.1 General

(1)   For a lattice structure of square or equilateral triangular plan form, having equal areas on each face, the total wind force coefficient c_f of a section in the direction of the wind:

$$c_{f,S} = K_\theta\, c_{f,S,0}$$ ... (B.2)

where: $c_{f,S,0}$ is the overall normal drag (pressure) coefficient of a section j without end-effects, determined in accordance with B.2.2.2;

K_θ is the wind incidence factor.

(2)   The wind incidence factor K_θ may be obtained from:

$$K_\theta \quad = 1{,}0 + K_1 K_2 \sin^2 2\theta \text{ for square structures} \qquad \text{... (B.3a)}$$

$$K_\theta \quad = \frac{A_c + A_{c,sup}}{A_S} + \frac{A_f}{A_S}\left(1 - 0{,}1\sin^2 1{,}5\theta\right) \text{ for triangular structures} \qquad \text{... (B.3b)}$$

with: $$K_1 \quad = \frac{0{,}55A_f}{A_S} + \frac{0{,}8\left(A_c + A_{c,sup}\right)}{A_S} \qquad \text{... (B.3c)}$$

$$K_2 \quad = 0{,}2 \text{ for } 0 \le \varphi \le 0{,}2 \text{ and } 0{,}8 \le \varphi \le 1{,}0 \qquad \text{... (B.3d)}$$

$$= \varphi \text{ for } 0{,}2 < \varphi \le 0{,}5 \qquad \text{... (B.3e)}$$

$$= 1 - \varphi \text{ for } 0{,}5 < \varphi < 0{,}8 \qquad \text{... (B.3f)}$$

in which: θ is the angle of incidence of the wind to the normal of face 1, in plan

φ is the solidity ration see 7.11 (2) of EN 1991-1-4.

A_f is the total projected area when viewed normal to the face of the flat-sided section members in the face

A_c is the total projected area when viewed normal to the face of the circular-section members in the face in subcritical regimes

$A_{c,sup}$ is the total projected area when viewed normal to the face, of the circular-section members in the face in supercritical regimes

h is the section height under consideration

b is the overall section width, as shown in Figure B.2.1
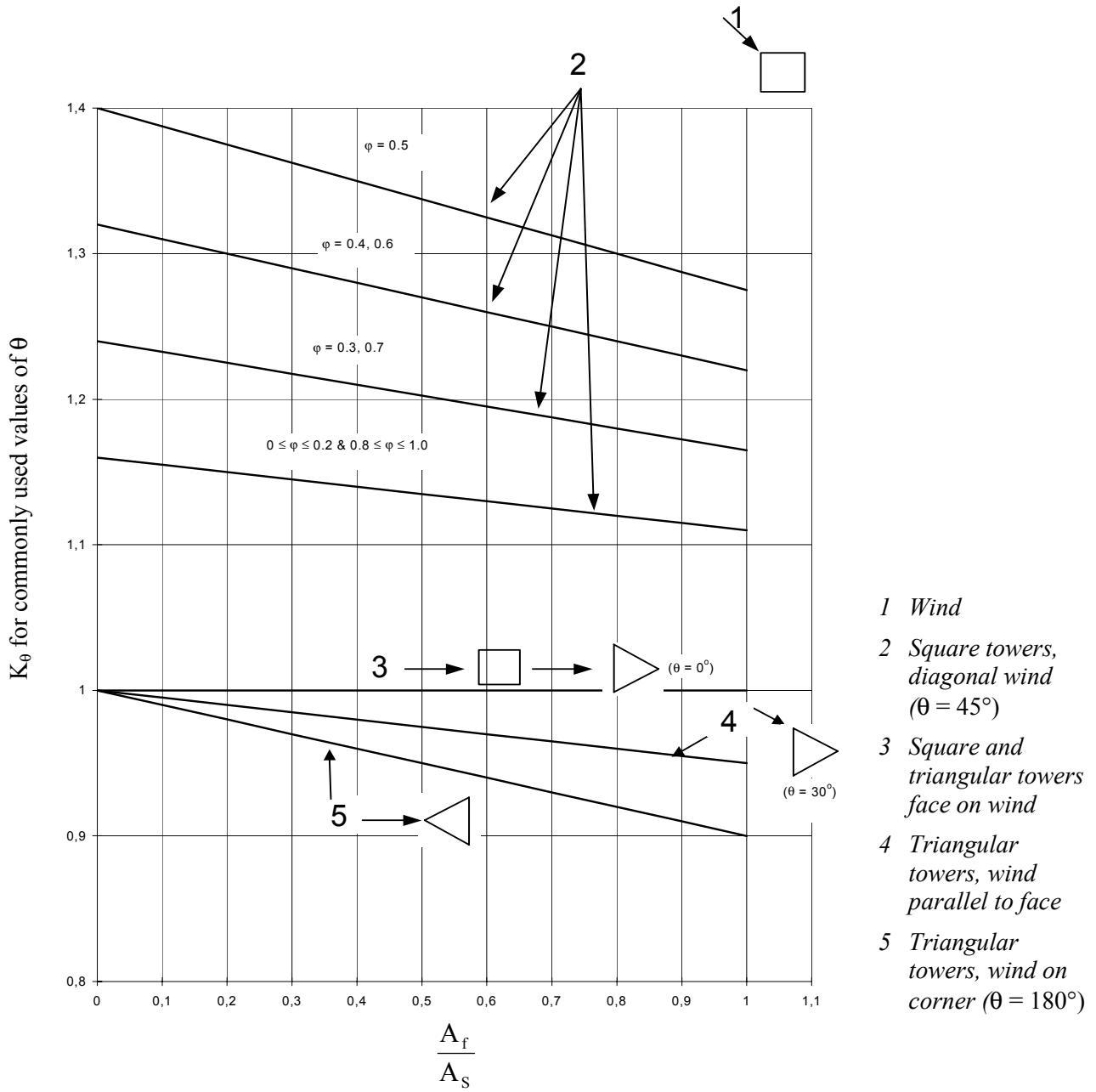
NOTE: $A_S = A_f + A_c + A_{c,sup}$

(3)   Values of K_θ for commonly used values of θ may be obtained from Figure B.2.2.

(4)   Circular-section members should be assumed to be in a subcritical regime when the effective Reynold's number $Re \le 4 \times 10^5$, and may be assumed to be in a supercritical regime for higher values of Re only when they are ice free.

(5)   The value of Re should be obtained from 7.9.1(1) of EN 1991-1-4.

(6)   Where supercritical flow is assumed for any or all members, it should be checked that greater loading does not result under a reduced windspeed corresponding to $Re < 4 \times 10^5$.

Figure B.2.2 Wind incidence factor K_θ

The graph shows:

- Y-axis: K_θ for commonly used values of θ, ranging from 0,8 to 1,4
- X-axis: $\dfrac{A_f}{A_S}$, ranging from 0 to 1,1

Curves labelled:
- φ = 0.5
- φ = 0.4, 0.6
- φ = 0.3, 0.7
- 0 ≤ φ ≤ 0.2 & 0.8 ≤ φ ≤ 1.0

(θ = 0°), (θ = 30°)

Legend:
1 Wind
2 Square towers, diagonal wind (θ = 45°)
3 Square and triangular towers face on wind
4 Triangular towers, wind parallel to face
5 Triangular towers, wind on corner (θ = 180°)

Key: For symbols see B.2.2.1

B.2.2.2 Overall normal force coefficients

(1)   Values of overall normal force coefficients $c_{f,S,0}$ that are applicable to the structural framework of a square or equilateral triangular section j composed of both flat-sided and circular-section members, should be taken as:

$$c_{f,S,0,j} = c_{f,0,f}\frac{A_f}{A_S} + c_{f,0,c}\frac{A_c}{A_S} + c_{f,0,c,\text{sup}}\frac{A_{c,\text{sup}}}{A_S}$$

... (B.4)

where: $c_{f,0,f}$, $c_{f,0,c}$ and $c_{f,0,c,\text{sup}}$ are the force coefficients for sections composed of flat-sided, subcritical circular and supercritical circular-section members, respectively, given by:

$c_{f,0,f}$ $=$ $1{,}76\, C_1 [1 - C_2\, \varphi + \varphi^2]$ ... (B.5a)

$c_{f,0,c}$ $=$ $C_1 (1 - C_2\varphi) + (C_1 + 0{,}875)\, \varphi^2$ ... (B.5b)

$c_{f,0,c,\text{sup}}$ $=$ $1{,}9 - \sqrt{\{(1 - \varphi)(2{,}8 - 1{,}14C_1 + \varphi)\}}$ ... (B.5c)

with: C_1 equal to: 2,25 for square structures.

1,9 for triangular structures.
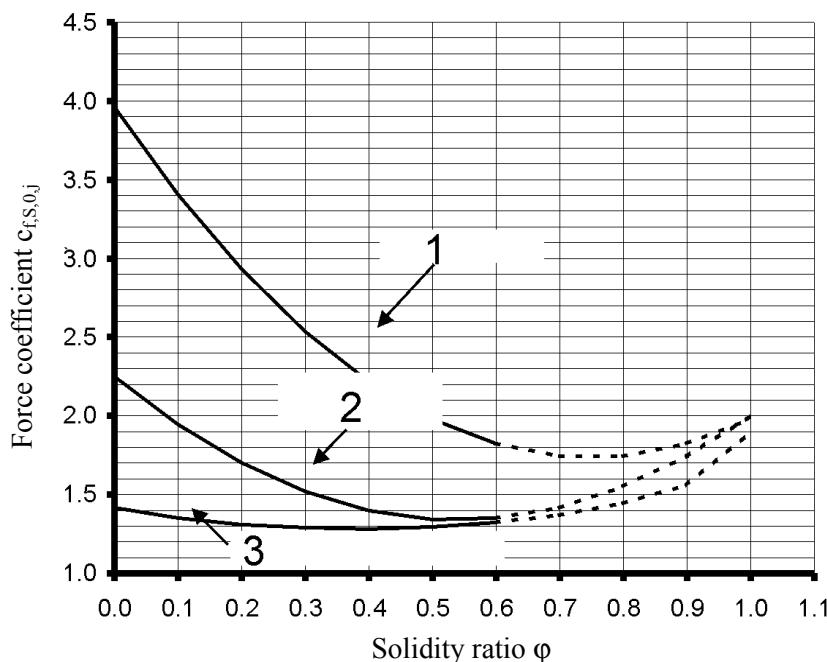
C_2 equal to: 1,5 for square structures;

1,4 for triangular structures.
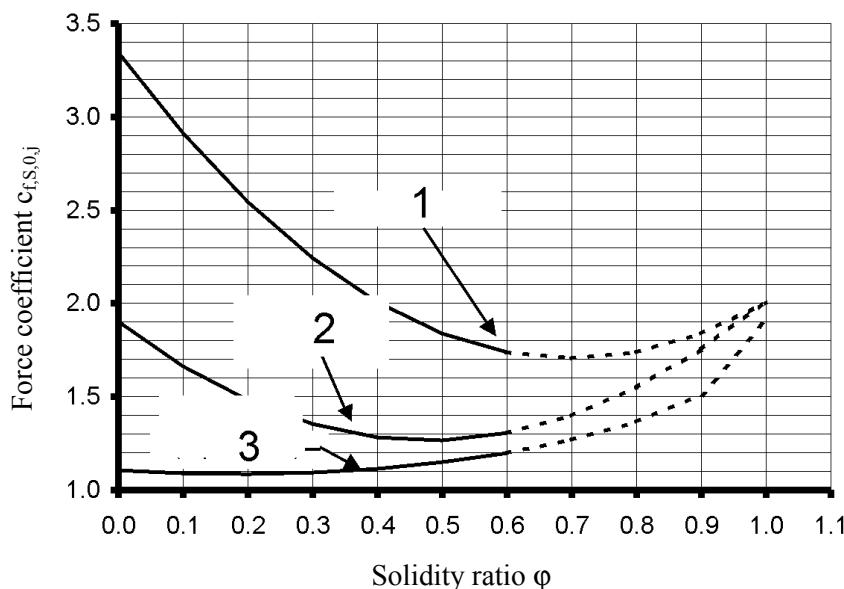
where: φ, A_S, A_f, A_c, $A_{c,\text{sup}}$ are defined in B.2.2.1

(2)   For force calculations circular section members in supercritical regimes may conservatively be assumed to be in subcritical regimes.

(3)   Approximate values of these force coefficients may be obtained from Figure B.2.3.

(a) Square structures

(b) Triangular structures

1 *Flat-sided*
2 *Circular (subcritical)*
3 *Circular (supercritical)*

NOTE: For structures with $\varphi > 0{,}6$ consideration should be given to the possibility of cross-wind response due to vortex excitation, see EN 1991-1-4.

Figure B.2.3 Overall normal force coefficients $c_{f,s,0}$ for square and triangular structures

B.2.3   Wind force coefficients of Linear ancillaries

(1)   The wind force coefficient $c_{f,A}$ in the direction of the wind of any linear ancillary part (including waveguides, feeders, etc.) within a panel height should be taken as:

$$c_{f,A} = c_{f,A,0} \, K_A \sin^2\psi \qquad\qquad\qquad ...\text{(B.6)}$$

where: $c_{f,A,0}$-   is the overall normal drag coefficient appropriate to the item and its effective Reynold's number, values of which are given in table B.2.1 for common isolated individual members and may be determined in accordance with B.2.7.2 for parts composed of single frames;

K_A is a reduction factor to take account of the shielding of the component by the structure itself and may only be taken into account when at least one face of the structure is effectively shielding the component (or vice versa); K_A is given in table B.2.2 except for circular sections in supercritical flow and for ancillaries not complying with the constraints of B.2.3 (2) in which case $K_A = 1.0$;

NOTE: Where A_A is greater than A_S the reduction factor should be applied to $c_{f,S,0}$ rather than $c_{f,A}$. Thus in such cases:

$$c_{f,S} = K_\theta \, c_{f,S,0} \, K_A$$

$$c_{f,A} = c_{f,A,0}\sin^2\psi$$

ψ is the angle of wind incidence to the longitudinal axis of any linear member.

(2) K_A should be taken as 1,0 for ancillary items that do not conform to any of the following restraints:

a) the total projected area of those ancillary parts adjacent to the face under consideration is less than the projected area of the structural members in that face (see Figure B.2.1);

b) the total projected area normal to any face on the structure of any single internal or external ancillary is less than half the gross area of the face of the panel; and

c) any ancillary does not extend more than 10% beyond the total face width of the structure at that level.

Table B.2.1 Typical force coefficients, $c_{f,A,0}$ and $c_{f,G}$, for individual components

Member type	Effective Reynold's number Re (see EN 1991-1-4) (see **NOTE 1**)	Drag (pressure) coefficient $c_{f,A,0}$ or $c_{f,G}$	
		Ice-free	Iced
(a) Flat-sided sections and plates	All values	2,0	2,0
(b) Circular sections and smooth wire	$\leq 2 \times 10^5$	1,2	1,2
	4×10^5	0,6	1,0
	$> 10 \times 10^5$	0,7	1,0
(c) Fine stranded cable, e.g. steel core aluminium round conductor, locked coil ropes, spiral steel strand with more than seven wires	Ice free: $\leq 6 \times 10^4$ $\geq 10^5$	1,2 0,9	
	Iced: $\leq 1 \times 10^5$ $\geq 2 \times 10^5$		1,25 1,0
(d) Thick stranded cable, e.g. small wire ropes, round strand ropes, spiral steel strand with seven wires only (1 x 7)	Ice free: $\leq 4 \times 10^4$ $> 4 \times 10^4$	1,3 1,1	
	Iced: $\leq 1 \times 10^5$ $\geq 2 \times 10^5$		1,25 1,0
(e) Cylinders with helical strakes of depth up to 0.12D (see **NOTE 2**)	All values	1,2	1,2
NOTE 1: For intermediate values of Re, $c_{f,A,0}$ should be obtained by linear interpolation. **NOTE 2:** These values are based on the overall width, including twice the strake depth. **NOTE 3:** The values for iced components are relevant for glazed ice; care should be exercised if they are used for rime ice (see ISO 12494). **NOTE 4:** These values may be changed in the National Annex.			

(3)   Where relevant, the corresponding torsional force T_{AW} should be calculated using the appropriate coefficient obtained from wind tunnel tests with the relevant moment arm for such torsion.

Table B.2.2 Reduction factor, K_A, for ancillary items

Position of ancillaries	Reduction factor, K_A	
	Square or rectangular plan form	Triangular plan form
Internal to the section	0,8	0,8
External to the section	0,8	0,8
NOTE : These values may be changed in the National Annex.		

B.2.4   Wind force coefficients of Discrete ancillaries

(1)   For any discrete ancillary item such as a dish reflector, the total wind force coefficient $c_{f,A}$ in the direction of the wind, should be taken as:

$$c_{f,A} = c_{f,A,0} \, K_A \qquad \qquad \text{... (B.7)}$$

where: $c_{f,A,0}$- is the drag coefficient for the item appropriate to the wind direction and wind speed and should be obtained from wind tunnel tests generally provided by the manufacturer;

   K_A is as defined in B.2.3.

(2)   The corresponding crosswind force coefficients $c_{f,A,x}$ and lift coefficient $c_{l,A,z}$ should be calculated as for $c_{f,A}$ taking the reference direction in plan as normal to the mean wind direction, and $c_{f,A,0}$ as the appropriate coefficient for crosswind and lift.

(3)   The corresponding torsional force coefficient T_{AW} should be calculated using the appropriate coefficient, obtained from wind tunnel tests in association with the relevant moment arm for such torsion.

B.2.5 Wind force coefficients of Guys

(1)   The wind force coefficient $c_{f,G}$ normal to the guys in the plane containing the guy and the wind should be taken as:

$$c_{f,G} = c_{f,G,0} \sin^2 \psi \qquad \qquad ... (B.8)$$

where: $c_{f,G}$ is the overall normal drag coefficient appropriate to the effective Reynold's number, the values of which are given in table B.2.1 for both ice-free and iced conditions;

   ψ is the angle of wind incidence to the chord.

   NOTE: The drag of guy insulators, where relevant, should be accounted for, either by using their appropriate drag as individual elements along the guy, or by smearing their effect into R_G.

B.2.6 Wind force coefficients under Iced conditions

(1)   In determining the wind resistance of a structure and ancillaries under iced conditions, each element of the structure, ancillary parts and guys should be taken as coated on all sides by ice, with a thickness equal to that given in Annex C.

(2)   Where the gap between components when not iced, is less than 75 mm, this should be assumed to be completely filled by ice under icing conditions.

(3)   Force coefficients of individual members should be obtained from table B.2.1.

(4)   Consideration should be given to asymmetric ice in which some guys are iced and some are ice-free (see Annex C).

B.2.7 Guidance for special cases and for the assessment of existing structures

B.2.7.1 Total wind force coefficient

(1)   The total wind force coefficient Σc_f in the direction of the wind over a panel height of an existing square or triangular structure that is being assessed or of a structure of rectangular unequal sided cross-section may be determined from (2) below.

   NOTE: For the design of square or equilateral triangular structures the method given in B.2.1.3 should be used.

(2)   The total wind force coefficient, Σc_f, in the direction of the wind over a panel height may be determined as follows:

–   for square and rectangular structures:

$$\Sigma c_f = c_{le} \cos^2 \theta_1 + c_{2e} \sin^2 \theta_1 \qquad \qquad ... (B.9)$$

–   for triangular structures:

$$\Sigma c_f = c_{le} \cos^2 \left(\frac{3\theta_1}{4} \right) + c_{2e} \sin^2 \left(\frac{3\theta_1}{4} \right) \qquad \qquad ... (B.10)$$

where: c_{le} is an effective wind force coefficient given by the following:

   –   for square and rectangular structures:

   $$c_{le} = (c_1 + \eta_1 c_3) K_{\theta 1}$$

   –   for triangular structures:

   $$c_{le} = \left\{ c_1 + \frac{\eta_1}{2} (c_2 + c_3) \right\} K_{\theta 1}$$

c_{2e} is an effective wind drag given by the following:

– for square and rectangular structures:

$$c_{2e} = (c_2 + \eta_2 \, c_4) \, K_{\theta2}$$

– for triangular structures:

$$c_{2e} = \left\{ c_2 + \frac{\eta_2}{2}(c_1 + c_3) \right\} K_{\theta2}$$

c_1 to c_4 are wind force coefficients given by:

$$c_1 \quad = \quad c_{f,S1} \, A_{s1}/\Sigma A \; + c_{f,A1} \, A_{A1}/\Sigma A;$$

$$c_2 \quad = \quad c_{f,S2} \, A_{s2}/\Sigma A \; + c_{f,A2} \, A_{A2}/\Sigma A;$$

$$c_3 \quad = \quad c_{f,S3} \, A_{s3}/\Sigma A + c_{f,A3} \, A_{A3}/\Sigma A;$$

$$c_4 \quad = \quad c_{f,S4} \, A_{s4}/\Sigma A \; + c_{f,A4} \, A_{A4}/\Sigma A;$$

A_{s1} to A_{s4} are the areas projected normal to faces 1, 2, 3 and 4, respectively, of the components treated as structural members within the same panel height of faces 1, 2, 3 and 4 including icing, where appropriate (see Figure B.2.1);

$c_{f,S1}$ to $c_{f,S4}$ are the force coefficients appropriate to faces 1 to 4, respectively, of the components treated as structural members which may be determined in accordance with B.2.7.2;

$c_{f,A1}$ to $c_{f,A4}$ are the wind force coefficients appropriate to faces 1 to 4, respectively, for the ancillary items not treated as structural members, determined in accordance with B.2.3 or B.2.4, as appropriate but taking $K_A = 1,0$ in all cases;

η_1 and η_2 are the effective shielding factors for faces 1 and 2, respectively, including both structural and ancillary components.

– for square structures η_1 and η_2 should be taken as: η_e

– for triangular structures η_1 and η_2 should be taken as: $0,67 \, \eta_e$

– for rectangular structures η_1 and η_2 should be taken as: $\eta_e + 0,15(\omega-1)(\varphi-0,1)$

but not greater than 1,0

η_e $= \eta_f(A_f + 0,83 \, A_c + 2,1 \, A_{c,sup} + A_A)/(A_s + A_A)$ but not greater than 1,0;

η_f is given by: $\eta_F = (1-\varphi)^{1,89}$ and is plotted in Figure B.2.4

where: $A_f, A_c, A_{c,sup}$ are as defined in B.2.2.1 applicable to faces 1 or 2, as appropriate;

A_A is the projected area normal to the face of the ancillary items not treated as structural members applicable to faces 1 to 4, as appropriate;

φ is the solidity ratio appropriate to face 1 or 2, as defined in Figure B.2.2, but including both structural and ancillary components

Thus $\varphi = \dfrac{A_S + A_A}{hb}$

ω is the spacing ratio for rectangular structures, equal to the distance between the face considered and that parallel to it divided by the width of the face considered at the level of the centroid of the panel area but not to be taken as less than 1,0;

$K_{\theta1}$ and $K_{\theta2}$ are to be obtained from B.2.2.1, applicable to faces 1 or 2, as appropriate, using $(A_S + A_A) \, A_f$ and φ as defined in this subclause;

θ is the plan angle of incidence of wind to the normal to face 1.

(3)   For structures with $\varphi > 0.6$ consideration should be given to the possibility of cross-wind response due to vortex excitation, see EN 1991-1-4.

(4)   The total crosswind force coefficients over a panel c_{fx} should be determined as in (2), but taking the reference direction as normal in plan to the mean wind direction.

(5)   The total wind force coefficient, Σc_f, in the direction of the wind over a panel height of polygonal shaped structures (with greater than four faces) should be determined from properly scaled wind tunnel tests in accordance with 1.5 of EN 1991-1-4.
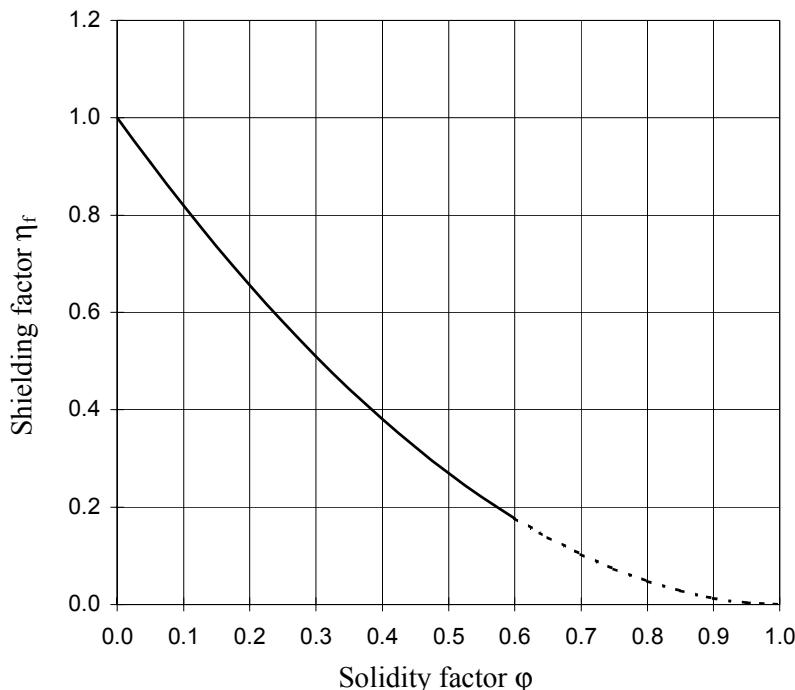

Figure B.2.4 Shielding factor η_f for single frames composed of flat-sided members

B.2.7.2 Wind force coefficients for single frames

(1)   Values of normal force coefficients c_f for single frames composed of both flat-sided and circular-section members should be taken as:

$$c_f = c_{f,f} \frac{A_f}{A_S} + c_{f,c} \frac{A_c}{A_S} + c_{f,c,\text{sup}} \frac{A_{c,\text{sup}}}{A_S} \qquad \ldots \text{(B.11)}$$

where: $c_{f,f}$, $c_{f,c}$ and $c_{f,c,\text{sup}}$ are the normal force coefficients for flat-sided, subcritical circular- and supercritical circular-section members, respectively, given by:

$c_{f,f}$ is the force coefficient for single frames equal to:

$1,58 + 1,05\,(0.6 - \varphi)^{1,8}$ for $\varphi \leq 0,6$;
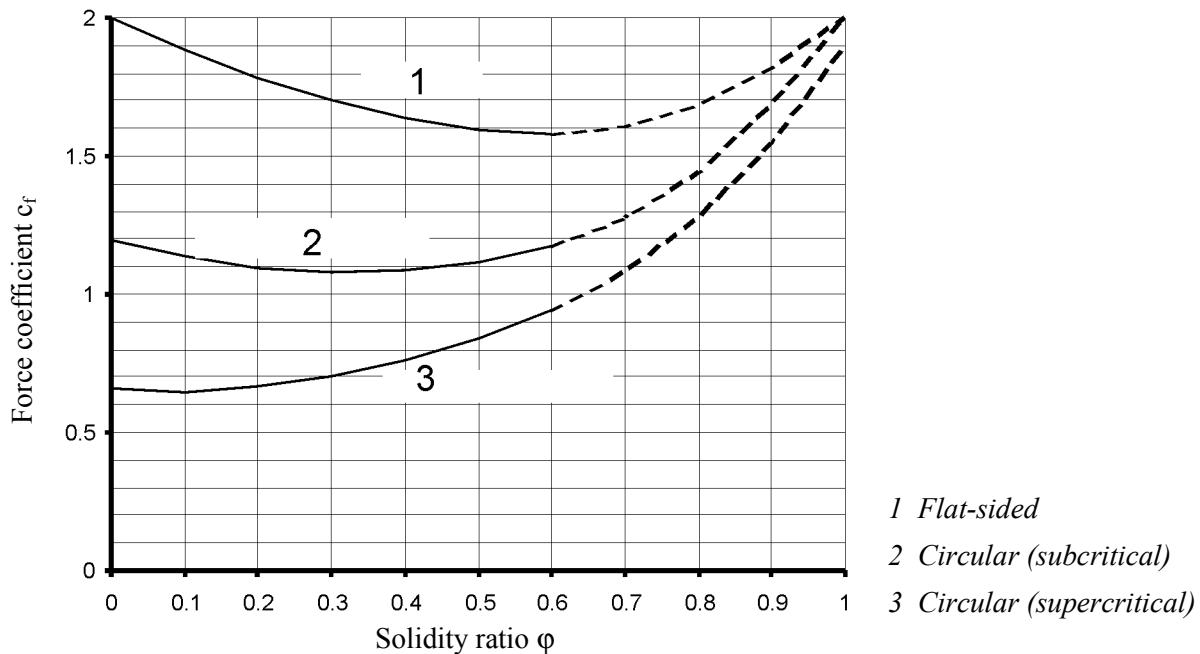
$1,58 + 2,625\,(\varphi - 0,6)^{2}$ for $\varphi > 0,6$;

A_f, A_c, $A_{c,\text{sup}}$, A_S and φ are as defined in B.2.7.1.

$c_{f,c}$ $= (0,6 + 0,4\varphi^2)\,c_{f,f}$

$c_{f,c,\text{sup}}$ $= (0,33 + 0,62\varphi^{5/3})\,c_{f,f}$

(2)   Approximate values of these drag coefficients are given in Figure B.2.5.

NOTE: For structures with $\varphi > 0{,}6$ see B.2.7.1(3).

Figure B.2.5 Normal forcecoefficient c_f for single frames

B.3 Response of lattice towers

B.3.1 Criteria for static methods

(1)   The equivalent static method, see B.3.3, should usually be used if the criteria in B.3.2(3) are met. If not, more complex methods such as the spectral analysis method, see B.3.4, should be used. Specialist advice is necessary.

(2)   The equivalent static method includes an allowance for the dynamic amplification of response that is typical of the majority of towers likely to be constructed in accordance with this standard. The check for applicability of the static procedure according to equation B.12 should be considered for guidance only. Dynamic augmentation generally increases in successively higher panels of any tower, particularly when supporting large concentrations of ancillary items or when using a concave outline profile (Eiffelization). In such cases caution should be exercised in applying the static procedure to towers where these effects are considerably more than those typically encountered.

(3)   The equivalent static procedures may be used if:

$$\frac{7m_T}{\rho_s c_{fT} A_T \sqrt{d_B \tau_o}} \left(\frac{5}{6} - \frac{h_T}{h} \right)^2 < 1 \qquad \qquad ... \text{(B.12)}$$

where: $c_{f,T}$ is the sum of the panel wind forces (including ancillaries), commencing from the top of the tower, such that $c_{f,T}$ is just less than one-third of the overall summation Σc_f for the whole tower (in m^2);

ρ_s is the density of the material of the tower structure (in kg/m^3);

m_T is the total mass of the panels making up $c_{f,T}$ (in kg);

h is the height of the tower (in m);

h_T is the total height of the panels making up $c_{f,T}$ but not greater than h/3 (in m);

τ_o is a volume/resistance constant taken as 0,001 m;

d_B is the depth in the direction of the wind, equal to:

–   base d for rectangular towers (in m);

–   0,75 x base width for triangular towers (in m).

B.3.2   Equivalent static method

B.3.2.1 General

(1)   For symmetrical towers constructed of leg members with triangulated bracings, with or without ancillaries for which the resistance has been calculated by B.2.2, maximum member forces should be derived in accordance with B.3.3.2.1 to B.3.3.2.7. For unsymmetric towers constructed of leg members with triangulated bracings and containing ancillaries, or for towers for which the resistance has been calculated by B.2.7 the maximum member forces should be determined in accordance with B.3.3.2.8.

NOTE: For symmetric triangular and square towers the wind loads in the cross-wind direction will not govern design and may thus be ignored. For unsymmetric towers these loads are taken into consideration.

B.3.2.2 Wind loading

B.3.2.2.1   General

(1)   The wind force in the direction of the wind on the tower should be determined with (5.4) of EN 1991-1-4:

B.3.2.2.2   Loading for calculating member forces or foundation forces

(1)   The maximum member force S_{max}, or forces on foundations should be determined from F_W and increased by a factor:

$$S_{max} = S_W \left(1 + \frac{c_s c_d - 1}{c_s c_d} 0.2 \left(\frac{z_m}{h} \right)^2 \right) \qquad \text{... (B.14)}$$

where: S_W is the member force or foundation force determined from the wind load F_W.

z_m is the height above the base at which the load effect is required;

h is the overall tower height;

B.3.2.2.3   Loading for calculating shear forces

(1)   The loading to be used to calculate bracing member forces should be based on the configuration of the tower.

NOTE: Shear forces on foundations are determined from B.3.3.2.2.

(2)   For towers in which the leg slopes are such that, when projected, they intersect above the top of the tower (see Figure B.3.1(a)) the maximum bracing force, or shear above a given level should be determined from B.3.3.2.2.

NOTE: Forces in bracing members at leg slope changes may have components from the leg force and from the shear.

(3)   For towers in which the legs in the panel being considered are inclined such that, when projected, they intersect below the height of the tower (see Figure B.3.1(b)), two 'patch' loading analyses should be undertaken with:

a) the mean wind loading, $F_{m,TW}(z)$, considered below the intersection and an equivalent 'gust' wind load above the intersection taken as $F_W(z) (1 + (c_s c_d - 1)/c_s c_d\ 0.2\ (z/h)^2)$;

45

$$F_{m,W} = \frac{F_W}{c_s c_d}$$
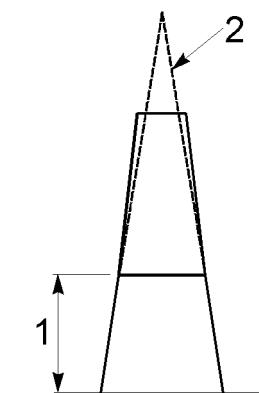
b) the mean wind loading considered above the intersection and an equivalent 'gust' wind load below the intersection taken as $F_W(z)$ $(1 + (c_s c_d - 1)/c_s c_d \ 0.2 \ (z/h)^2)$.

where: F_W is the wind loading taken from EN 1991-1-4
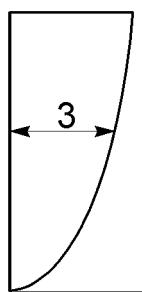
      $c_s c_d$ is the structural factor from EN 1991-1-4

(4)   For more than one such intersection, two patch loading cases should be analysed for each panel, see Figure B.3.1(c).
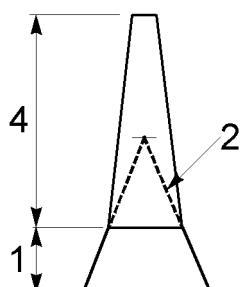
> **NOTE:** For bracing members above the highest intersection point the procedure of B.3.3.2.3(2) may be used.

$$F_{m,W} = \frac{F_W}{c_s c_d}$$
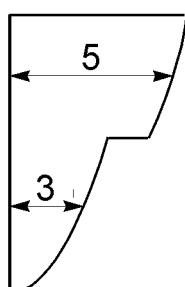
(a) Case 1
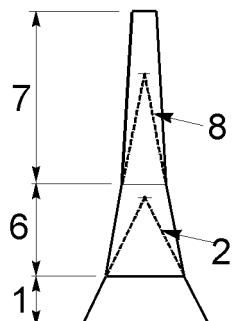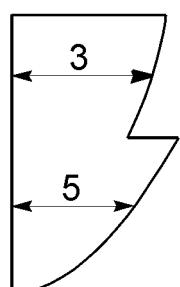
All shears determined from mean loading and gust response factor
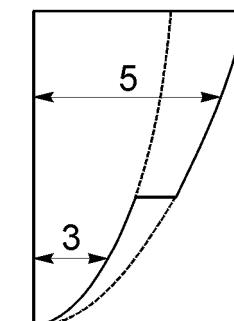
(b) Case 2
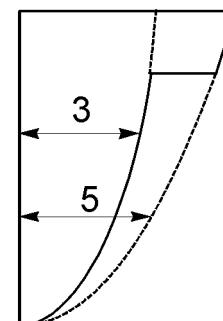
Patch loading for panel "A"

(c) Case 3

Patch loading for panel "A":
patch 1 ————
patch 2 – – – – –

Patch loading for panel "B":
patch 1 ————
patch 2 – – – – –

1 Panel "A"

2 Projection of legs from panel "A"

3 mean

4 Panel "A" as case 1, treat panels above

5 "gust"

6 Panel "B"

7 Panel "B" as case 1, treat panels above

8 Projection of legs from panel "B"

Figure B.3.1 Shear patch loading

B.3.2.2.4   **Loading on cables and guys supported by the tower**

(1)   The maximum wind loading on cables and guys in the direction of wind $F_{c/Gw}(z)$ should be taken as:

$$F_{cw}(z) = q_m(z)\sum c_{f,G} \cdot A_G \cdot c_s c_d \qquad \text{... (B.16)}$$

where:

$q_m(z)$ is the mean wind pressure at the effective height of the cable, z, metres above site ground level determined in accordance with EN 1991-1-4;

$\Sigma c_{f,G}$ is the total wind force coefficient on the guy/cable in the direction of the wind, determined in accordance with B.2;

$c_s c_d$ is a structural factor for cables in accordance with EN 1991-1-4

B.3.2.2.5   Loading for calculating deflections and rotations

(1)   Deflections and rotations are normally only important to satisfy serviceability requirements. The serviceability criteria should be defined by the client in the project specification (see 7.2.2).

B.3.2.2.6   Wind loading for unsymmetrical towers or towers with complex attachments

(1)   For unsymmetrical towers or towers that contain unsymmetrically placed large ancillaries and/or cables imposing significant torsional and crosswind loads, the total forces due to the effect of wind load should allow for the combined action of wind on individual parts, both along wind and crosswind, when appropriate.

(2)   The fluctuating load effects caused by cross wind turbulence should be considered in conjunction with along wind load effects.

(3)   To determine the total load effects in such cases the mean along wind load effect should be separated from the fluctuating wind load effect. Thus the tower should be analysed under the mean wind load in the direction of the wind ($F_{m,W}(z)$) as determined from B.3.3.2.1(1).

> **NOTE:** If cables are present the mean load on the cables $F_{m,CW}(z)$ should be used (see B.3.3.2.8 (4) (d)).

(4)   The individual load effects should then be calculated as:

a) the mean wind load effect, $S_{m,TW}$, determined from the mean wind load $F_{m.TW}(z)$.

b) the fluctuating in line wind effect, $S_{1,TW}$, determined from:

$$S_{1,TW} = S_{m,TW} \left(1 + (c_s c_d - 1)/c_s c_d \, 0.2 \, (z/h)^2\right) \qquad \text{... (B.17)}$$

c) Turbulence in the crosswind direction causes fluctuating crosswind load effects ($S_{1,TX}$) which, in the absence of other information should be taken as:

$$S_{1,TX} = K_X \left(\frac{\sum c_X}{\sum c_f} \right) S_{1,TW} \qquad \text{... (B.18)}$$

where: K_X is a factor to allow for crosswind intensity of turbulence.

Σc_X is the crosswind lift coefficient of the structure (and any ancillaries if present) over the panel height concerned.

> **NOTE 1:** The value of K_X may be given in the National Annex. The value $K_X = 1,0$ is recommended.

> **NOTE 2:** Crosswind turbulence will cause fluctuating crosswind loads even in symmetric towers; however such loads will not normally affect the critically loaded elements.

(5)   The total load effect ΣS_T in any member due to wind should then be taken as:

$$\sum S_T = S_{m,TW} + S_{m,cw} + \sqrt{S_{1,TW}^2 + S_{1,TX}^2 + S_{cables}^2} \qquad \text{... (B.20)}$$

B.3.4   Spectral analysis method

(1)   When response to along wind forces is calculated by a spectral analysis, the meteorological conditions to be assumed should be those defined in EN 1991-1-4, and the wind resistance taken as that given in B.2. In addition, the parameters defined in Annex B of EN 1991-1-4 should be adopted in the absence of more accurate information.

> **NOTE:** The National Annex may give further information.

(2)   Cross wind turbulence will cause fluctuating load effects which need to be considered in conjunction with in-line wind loads. Appropriate parameters, consistent with those adopted for downwind effects should be adopted.

> **NOTE:** The National Annex may give further information.

B.3.5   Crosswind vibrations

(1)   If towers support large prismatic, cylindrical or bluff bodies or may be expected to become heavily blocked by icing, their susceptibility to vortex-excited vibrations and/or galloping should be determined, in accordance with EN 1991-1-4.

B.4   Response of guyed masts

B.4.1   General

(1)   The maximum forces to be used in the design of mast components and foundations should be calculated with due allowance for the response to wind turbulence.

(2)   Such forces should represent the resultant effect of an equivalent static loading due to wind of speed equal to the appropriate 10 minute mean value, acting only in the wind direction, and fluctuating loading both downwind and, where relevant, crosswind due to gustiness.

B.4.2   Criteria for static methods

(1)   Generally static analysis procedures can be used to determine the maximum forces in the members of a mast (see B.4.3). Only for masts which may be prone to significant dynamic response is it necessary to undertake dynamic response methods (see B.4.4).

(2)   The design of major masts whose economic consequences of failure or potential hazards resulting from failure are high (see 2.3) should be checked by dynamic response procedures if required by the project specification.

(3)   The following criteria should be satisfied for the static analytical procedures to be used:

a) any cantilever has a total length above the top guy level of less than half the spacing between the penultimate and top guys;

b) the parameter β_s is less than 1, where:

$$\beta_s = \frac{4\left(\dfrac{E_m I_m}{L_S^2}\right)}{\left(\dfrac{1}{N}\displaystyle\sum_{i=1}^{N} K_{Gi} H_{Gi}\right)} \qquad \text{... (B.21a)}$$

with: $K_{Gi} = 0,5 N_i A_{Gi} E_{Gi} \cos^2\alpha_{Gi}/L_{Gi}...$ (B.21b)

where: N   is the number of guy levels;

A_{Gi} is the cross sectional area of guy at level i;

E_{Gi} is the elastic axial modulus for guy at level i;

L_{Gi} is the length of guy at level i;

N_i is the no. of guys attached at level i;

H_{Gi} is the height above the mast base of the ith guy level;

α_{Gi} is the slope of the guy chord at level i to the horizontal;

E_m is the elastic modulus for the mast;

I_m is the average mast bending inertia;

L_s is the average span between guy levels.

c) The parameter Q is less than 1, where:

$$Q = \frac{1}{30} \sqrt[3]{\frac{HV_H}{D_o}} \sqrt{\frac{m_o}{HR}} \qquad \qquad \text{... (B.21c)}$$

m_o is the average mass per unit length of the mast column including ancillaries (kg/m);

D_o is the average face width of the mast (m);

V_H is the mean wind speed V_e at top of mast (m/sec);

R is the average total wind drag obtained from A.2 (m^2/m);

H is the height of mast, including cantilever if present (m).

(4)   If any of the criteria in (3) are not satisfied, then the spectral analysis method (see B.4.4) should be followed.

B.4.3   Equivalent static methods

B.4.3.1 General

(1)   To allow for the dynamic response of masts to wind loading the mast should be analysed for a series of static 'patch' loading patterns based on the mean loading augmented by wind load 'patches'. This procedure requires several static wind analyses for each wind direction considered, the results being combined to provide the maximum response.

(2)   For masts of symmetrical structural cross section with triangulated bracing, either without ancillaries or with ancillaries symmetric in the wind direction being considered, and are not likely to be dynamic sensitive (see B.4.7), the maximum forces should be derived in accordance with B.4.3.2.

(3)   For masts containing ancillaries which are unsymmetric in the wind direction being considered, the additional forces due to cross wind effects should be determined in accordance with B.4.3.2.8.

B.4.3.2 Load cases to be considered

B.4.3.2.1   Mean wind loading

(1)   The wind load in the direction of the wind on the mast column F_{MW} due to the mean wind speed should be taken as:

$$F_{MW} = \frac{\rho}{2} V_m^2(z) \sum R_W(z) \qquad \qquad \text{... (B.22)}$$

where: ρ is the density of the air at the reference temperature and pressure;

        $V_m(z)$ is the mean wind speed at the level of the centre of area of the section at a height z metres above the site ground level, determined in accordance with 4.3 of EN 1991-1-4;

        $\sum R_W(z)$ is the total wind drag of the structure (and any ancillaries if present) in the direction of the wind over the mast section concerned, at a height z metres above the site ground level, determined in accordance with B.2.

(2)   Unless otherwise specified in the National Annex of EN 1991-1-4 the value of ρ should be taken as 1,25 kg/m^3.

(3)   The loads should be taken as acting at the level of the centre of areas of faces (including ancillaries if

present) within the section height.

(4) The wind loading on the guys, F_{GW}, normal to the guys in the plane containing the guy and the wind, due to the mean wind speed should be taken as:

$$F_{GW} = \frac{\rho}{2} V_m^2(z) R_G(z) \qquad \qquad \text{... (B.23)}$$

where: $R_G(z)$ is the wind drag of the guy under consideration determined in accordance with B.2;

$V_m(z)$ is the mean wind speed at each level with which the guy is subdivided for analysis.

(5) If a uniform loading is used then $V_m(z)$ should be taken as the wind speed at ⅔ the height of the relevant guy attachment to the mast.

(6) The load effects S_m due to the mean wind should be determined for each component of the mast by a geometric non-linear static analysis under the mean loading F_{MW} and F_{GW}.

B.4.3.2.2 Patch loads

(1) In addition to the mean loading derived from B.4.3.2.1 successive patch loads should be applied as follows:

– on each span of the mast column between adjacent guy levels (and the span between the mast base and the first stay level);

– over the cantilever if relevant;

– from midpoint to mid point of adjacent 'spans';

– from the base to the mid height of the first guy level;

– from the mid height of the span between the penultimate and top guy if no cantilever is present, but to include the cantilever if relevant.

(2) These are shown in Figure B.4.1. The 'patch' load should be taken as:

$$F_{PW} = \rho k_p I_v(z) \left(\frac{1}{C_0(z)} \right) V_m^2(z) \sum R_W(z) \qquad \qquad \text{... (B.24)}$$

where: ρ, $V_m(z)$ and $\Sigma R_W(z)$ are as defined in B.4.3.2.1;

k_p is a peak factor on wind turbulence as defined in 6.3.1 of EN 1991-1-4 and may be taken as $k_p = 3,5$;

$I_v(z)$ is the turbulence intensity as given in 4.4 of EN 1991-1-4, depending on the site terrain and the orography.

$C_o(z)$ is the orography factor determined from EN 1991-1-4.

NOTE: For simplicity uniform patch loads may be used taking z as the height at the top of the patch for $I_v(z)$ and $V_m(z)$.
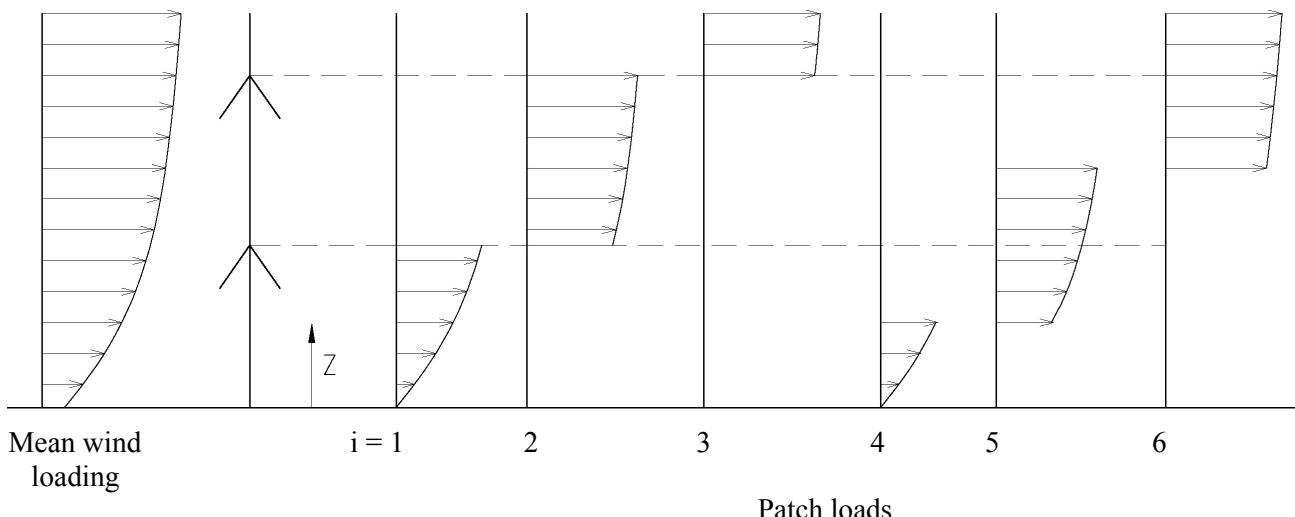
Figure B.4.1 Application of patch loads

(3)   These patch loads should be applied to the mast, under mean wind loading determined from B.4.3.2.1.

(4)   For masts up to 50m height only one case needs to be considered, with the mean and patch load enveloping the mast.

> **NOTE 1:** In such cases the shear bracing in each span should be designed for the maximum shear (and associated torsion) in that span.

> **NOTE 2:** In such cases the legs and their connections in each span should be designed for the maximum (and minimum) leg load in that span.

> **NOTE 3:** In such cases if the mast supports a cantilever, then (i) mean plus patch loading on the cantilever and mean load on the mast and (ii) mean load on the cantilever and mean plus patch loading on the mast should also be considered.

B.4.3.2.3   Loading on guys

(1)   For each patch loading case on the mast column, as given in B.4.3.2.2 patch wind loads, F_{PG}, should be applied within the same boundaries, see Figure B.4.2. These patch loads should be applied normal to each guy in the plane containing the guy and the wind, and taken as:

$$F_{PG} = \rho k_p I_v(z) \left(\frac{1}{C_0(z)} \right) V_m^2(z) R_G(z) \qquad \text{... (B.24)}$$

where: ρ, $V_m(z)$ are defined in B.4.3.2.1;

   $I_v(z)$ is defined in B.4.3.2.2;

   k_p is a peak factor on wind turbulence as defined in 6.3.1 of EN 1991-1-4, and may be taken as $k_p = 3,5$;

   $R_G(z)$ is the wind drag normal to the guy in the plane containing the guy and the wind determined in accordance with B.2.

(2)   For simplification the patch loading may be 'smeared' over the whole height of the guys in question by multiplying the above wind load by the ratio z_p/z_G:

where: z_p is the "height" of the patch on the actual guy; and

   z_G is the height to the attachment of the guy to the mast.
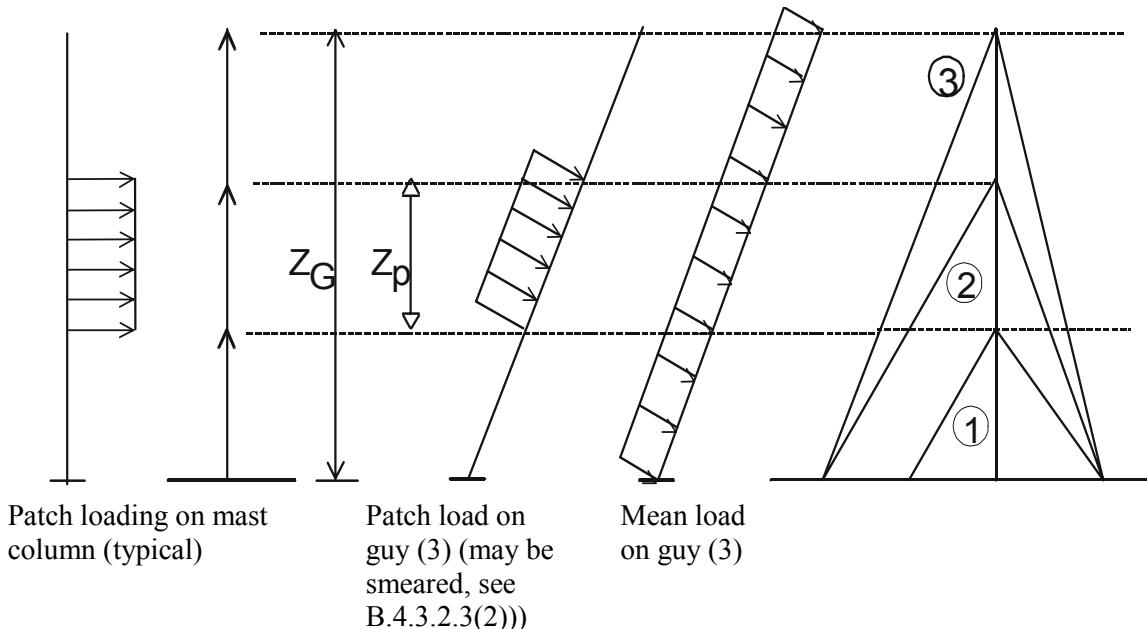
Figure B.4.2 Patch loading on guys

B.4.3.2.4   Derivation of response under patch loads

(1)   The load effect in each element of the mast column and guys derived from each patch load applied successively, S_{PLi}, should be calculated.

(2)   This should be done by calculating the difference between the load effect from the patch load combined with the mean load and the load effect of the mean load alone.

(3)   These load effects should then be combined as the root sum of squares, or:

$$S_p = \sqrt{\sum_{i=1}^{N} S_{PLi}^2} \qquad \qquad ...\text{(B.25)}$$

where: S_{PLi} is the load effect (response) from the ith load pattern;

   N   is the total number of load patterns required;

   S_p is the total effective load effect of the patch loads.

B.4.3.2.5   Total load effects
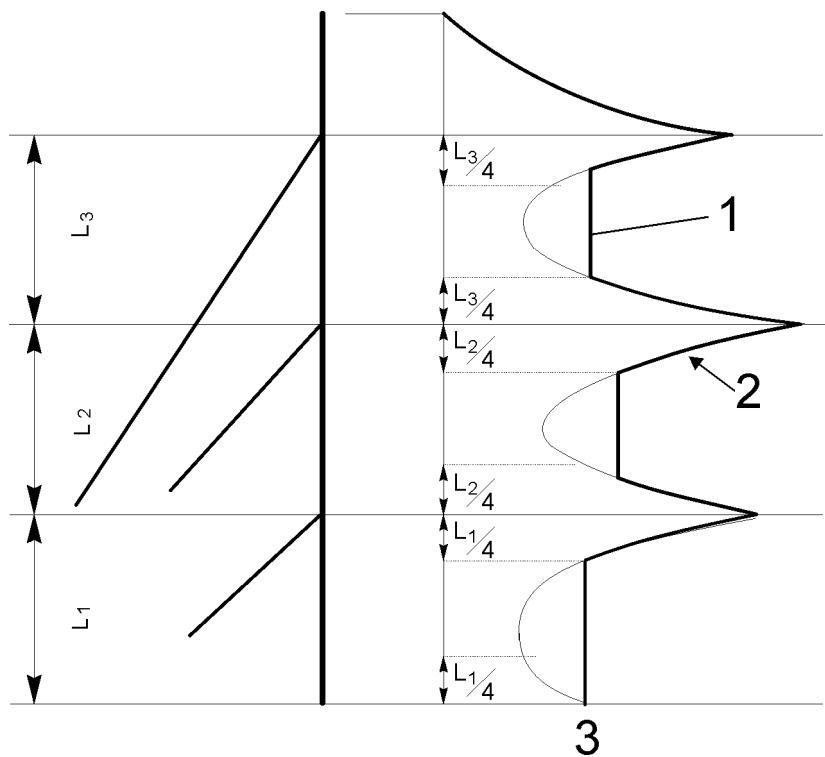
(1)   The total load effects for each component of the mast column, S_{TM}, should be determined from:

   $S_{TM} = S_M \pm S_p$ $\qquad \qquad$ ... (B.26)

where: S_M is the mean load effect determined from B.4.3.2.1;

   S_p is the fluctuating load effect determined from B.4.3.2.4 using the sign to produce the most severe effect.

(2)   In the calculation of the total force in the shear bracing in each span of the mast column in accordance with (1) above, the minimum value within that span should be taken as the highest calculated at a distance of one quarter of the span from either adjacent guy attachment levels (or the mast base if relevant). In this context 'span' refers to the distance between adjacent guy levels or between the base and the lowest guy level. (See Figure B.4.3.)

NOTE: Envelope of forces in bracing members arising from patch loading (absolute values shown)

Figure B.4.3 Minimum forces in shear bracing in mast column

B.4.3.2.6   Wind directions to be considered

(1)   For each member of the mast, the wind direction giving the most severe total load effect should be considered. This in practice means that several wind directions should be investigated.

(2)   If the mast is nearly symmetrical in geometry and loading, at least three wind directions should be analysed for a triangular mast guyed in three directions, i.e. normal , 30° to a face and 60° to a face. for a mast with square cross section and guyed in four directions, at least two wind directions should be analysed, normal to a face and 45° to a face. Examples are shown in Figure B.4.4.

> NOTE: To account for overall buckling of symmetric masts (see 5.1(5)) introduction of a lateral effect (such as a cross-wind force of 2% of the along wind force or a wind direction of 2° off the notional wind direction) should be provided in undertaking the second order analysis.

(4 stayed circular mast similar)
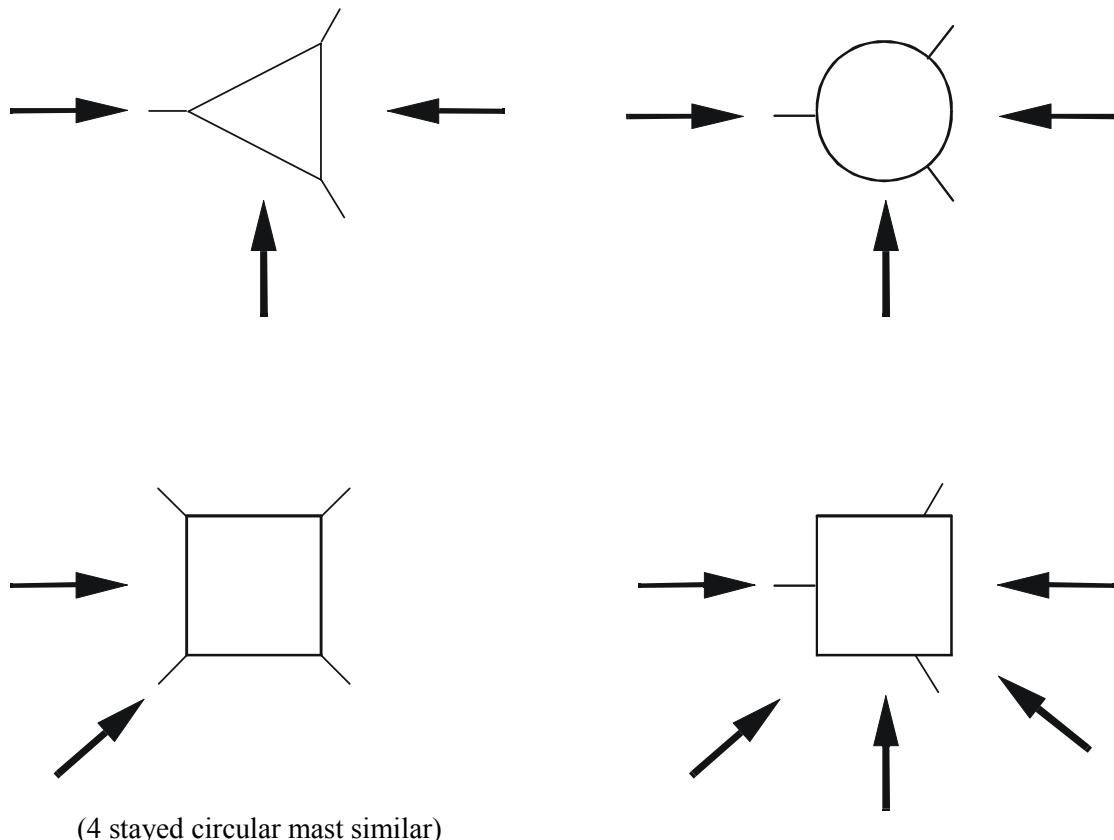
Figure B.4.4 Typical wind directions to be considered

B.4.3.2.7   Loading for calculating deflections and rotations

(1)   Deflections and rotations are normally only important to satisfy serviceability requirements. The serviceability criteria should be defined by the client in the project specification (see 7.2.2)

B.4.3.2.8   Wind loading for unsymmetrical masts or masts with complex attachments

B.4.3.2.8.1   General

(1)   For unsymmetrical masts or masts that contain unsymmetrically placed large ancillaries and/or cables imposing torsional and cross wind loads, the total forces due to the effects of wind load should allow for the combined action of wind on individual parts, both along wind and crosswind, when appropriate.

(2)   Cross wind turbulence will cause fluctuating load effects. This may need to be considered in conjunction with along wind loads.

(3)   The procedure for separating the mean along wind loads from the fluctuating loads needs to be carried out, as set out for towers in B.3.3.2.8. For guyed masts, however this will necessitate a series of transverse patch wind loads to be applied in a similar manner to those for along wind as set out in B.4.3.2.2.

(4)   The total load effects should then be determined from:

$$S_{TM} = S_M \pm \sqrt{S_{PW}^2 + K_X^2 S_{PX}^2} \qquad \qquad ... \text{(B.27)}$$

where: S_{PW} is the load effect from the in-line patch loads;

S_{PX} is the load effect from the cross-wind patch loads;

K_X is a factor to allow for cross wind intensity of turbulence

NOTE 1:   The value of K_X may be given in the National Annex. The value $K_X = 1,0$ is recommended.

NOTE 2: Cross wind turbulence will cause fluctuating cross wind loads even in symmetric masts; however such loads will not affect the critically loaded elements.

(5)   Alternatively, for simplification the cross wind turbulence effects need not be calculated explicitly as in B.4.3.2.8.1(4) above but the in-line peak load effects, S_{TM}, from B.4.3.2.5(1) should be increased by 10% to allow for cross wind effects.

B.4.4   Spectral analysis method

(1)   When response is calculated by spectral analysis this should be used for the resonance contribution to the response only.

(2)   The non-resonant response may be determined using the general static procedure (See B.4.3.2) using $k_p = 3,5$.

(3)   The meteorological conditions to be assumed should be those defined in EN 1991-1-4, and the wind resistance taken as that given in B.2. In addition, the parameters defined in Annex X of EN 1991-1-4 should be adopted in the absence of more accurate information.

(4)   Cross wind turbulence will cause fluctuating load effects which need to be considered in conjunction with along wind loads. Appropriate parameters, consistent with those adopted for along wind effects should be adopted.

(5)   Response should be calculated for all modes of vibration having natural frequencies less than 2 Hz.

B.4.5   Vortex-excited vibrations

(1)   When masts support large bluff bodies or are likely to become heavily blocked by icing, then susceptibility to vortex-excited vibrations, should be taken into account in accordance with EN 1991-1-4.

B.4.6   Guy vibrations

(1)   The mast guys should be checked for high frequency vortex-excited vibrations and guy galloping, particularly when the guys are iced, as follows:

a) *Vortex excitation*

Guys may be subject to low amplitude resonant type vibrations at low wind speeds caused by vortex excitation at high frequency.

As excitation can occur in high modes general rules cannot be set down. However as a guide, experience shows that such vibrations are likely to occur if the still air tensions in the guys are in excess of ten per cent of their breaking load.

b) *Galloping (including rain induced vibrations)*

Guys may be subject to galloping excitation when coated with ice or thick grease. The accretion of ice or grease can form aerodynamic shapes which provide lift and drag instabilities. These result in low frequency high amplitude vibrations. Similar vibrations are also known to occur under conditions of rain.

Again general rules cannot be provided as the occurrence of galloping is critically dependent on the formation of ice, or profile of grease. It will generally only occur on large diameter guys and is relatively insensitive to initial stay tensions. See EN 1993-1-11 (Clause 8.3)

(2)   If guy vibrations are observed, dampers or spoilers should be provided as required to limit the resulting stresses, see D.2.

(3)   Fatigue checks of the anchorages should be made if such vibrations are known to have occurred and no remedial action has been taken. In such cases specialist advice should be sought.

Annex C [informative] – Ice loading and combinations of ice with wind

NOTE: As this Annex deals with ice loading and combinations of ice with wind for masts and towers it is expected that it will be transferred to EN 1991 - Actions on structures.

C.1   General

(1)   Atmospheric ice loading on masts and towers can, for exposed sites, grow to considerable thicknesses, and combined with wind the increased wind drag due to iced members might in some instances govern the design.

(2)   The magnitude of ice deposit on structures, as well as the density, the placing and the shape of the ice on masts and towers heavily depends on the local meteorological conditions and the topography and the shape of the structure itself.

(3)   Atmospheric icing is traditionally classified according to two different formation processes:

–   in-cloud icing;

–   precipitation icing.

(4)   These may result in various types of ice as soft rime, hard rime, wet snow and glaze, having different physical properties concerning density, adhesion, cohesion, colour and shape. For instance the density varies typically from approximately 200 kg/m^3 to 900 kg/m^3, and from a concentric deposit (glaze and wet snow) to an eccentric deposit on one face pointing windward for soft and hard rime.

(5)   For engineering design purposes it is traditionally assumed that all members of a mast or a tower are covered with a certain ice thickness, which together with a density may be used for calculation of the weight of the ice as well as the wind drag. Such methods may be justified in areas where glaze or wet snow form the design ice load, but in the case of rime the physical reality does not coincide with a uniform ice thickness on all members of towers and masts. However in areas where the ice deposit from in-cloud rime is relatively small, the method of calculating ice weight and wind drag with ice assuming a uniform ice cover can be practical and reasonable if conservative values are used.

(6)   On the other hand there are areas in Europe that are very exposed to heavy atmospheric icing and for such areas the ice load should be estimated by experts in atmospheric icing. This should include the weight, the location, the shape, etc. of the ice load on the actual structure, as well as the appropriate combination of ice with wind which should be specified in detail.

(7)   The following clauses provide a general description of how to treat ice load and ice in combination with wind on towers and masts.

C.2   Ice loading

(1)   The principles for characteristic ice loading inclusive of the density and other design parameters is given in ISO 12494. In ISO 12494 the ice load is based on Ice Classes for rime and glaze, but the actual Ice Class for the location is not given, nor is the density of the ice.

NOTE: The National Annex may give further information.

(2)   As the ice may deposit asymmetrically on towers and masts, such situations should be taken into account. Asymmetrical icing is of particular interest for masts where icing on the different guys may vary considerably causing bending effects in the mast column. Asymmetrical ice on the guys may partly be caused by asymmetrical ice accretion depending on wind direction and partly caused by unequal shedding of ice from the guys.

(3)   For lattice towers the weight of the ice itself does not normally govern the design even if the ice deposits asymmetrically on the tower.

C.3   Ice weight

(1)   When estimating the weight of the ice on a lattice tower or mast column, it may normally be assumed that all structural members, components of ladders, ancillaries, etc. are covered with ice having the same thickness over the whole surface of the member, see Figure C.1.
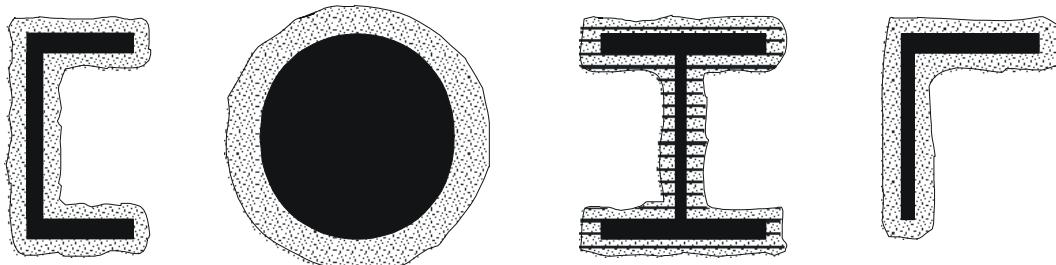

Figure C.1 Ice thickness on structural members

C.4   Wind and ice

(1)   In areas where atmospheric icing occurs, combinations with wind can often govern the design of masts and towers. The increased wind drag caused by the ice deposit on the individual members might thus result in critical loading, even though the associated wind speeds are less than the maximum characteristic values.

(2)   The wind drag of an iced tower or mast may for glaze ice be estimated using the same basic procedure as given in Annex B, taking into account the increased width of members and component due to the ice thickness. If the gaps between elements are small (say less than 75mm) they may be assumed to be closed by ice. For rime ice estimation of the wind drag is far more complicated, and for fully iced mast or mast faces special attention should be taken. Guidelines are given in ISO 12494.

(3)   When combining ice and wind load, the characteristic wind pressure in periods where atmospheric icing can occur is less than the characteristic wind pressure in all situations. This may be taken into account by multiplying the characteristic wind pressures in EN 1991-1-4 by a factor k. The factor k is given in ISO 12494 dependent of the Ice Class.

C.5   Asymmetric ice load

(1)   Asymmetric icing on a mast should be taken into consideration by applying the appropriate ice to the mast shaft and to all guys <u>apart</u> from:

–   one lane of the top guy level; and as a separate case;

–   two lanes of the top guy level.

C.6   Combinations of ice and wind

(1)   Two combinations of wind and ice should be taken into consideration for both symmetrical icing and asymmetrical icing. The design values of the loads are as given in 2.3 and the following combinations should be used:

–   for dominant ice and accompanying wind:

$$\gamma_G\, G_k + \gamma_{ice}\, Q_{k,ice} + \gamma_W\, k\, \psi_W\, Q_{k,w} \qquad\qquad \text{... (C.1)}$$

–   for dominant wind and accompanying ice:

$$\gamma_G\, G_k + \gamma_W\, k\, Q_{k,w} + \gamma_{ice}\, \psi_{ice}\, Q_{k,ice} \qquad\qquad \text{... (C.2)}$$

NOTE: The National Annex may give information on combination factors. The following combination factors are recommended:

$$\varphi_W \quad = \quad 0{,}5 \qquad\qquad\qquad\qquad\qquad \text{... (C.3a)}$$

$$\varphi_{ice} \quad = \quad 0{,}5 \qquad\qquad\qquad\qquad\qquad \text{... (C.3b)}$$

(3)   For partial factors on dead load γ_G, ice load γ_{ice} and wind load γ_W see Annex A.

Annex D [normative] – Guys, dampers, insulators, ancillaries and other items

D.1   Guys

D.1.1   Metallic guys and tension elements

(1)   For metallic guys and tension elements see EN 1993-1-11.

   NOTE: The National Annex may give further information.

D.1.2   Non metallic guys

(1)   Materials other than steel may be used provided that they have an acceptable modulus of elasticity and provided that appropriate measures are taken to prevent vibrations in higher frequencies.

   NOTE: In the selection of Synthetic materials the low modulus of elasticity of some products may require a higher initial tension to compensate for their lower stiffness, which can lead to possible high frequency vibrations. The ends of such ropes are sealed to prevent entrance of moisture which might otherwise lead to the discharge of lightning. Partial factors for non metallic guys may need to be higher than for steel guys.

(2)   Non metallic guys should comply with the relevant technical specification.

   NOTE: The National Annex may give further information.

D.2   Dampers

D.2.1   Structure dampers

(1)   The possible structural vibrations that can occur in a tower or mast under windshould be reduced, if necessary, by the use of the damping devices.

   NOTE: See Annex B and Annex A of EN 1993-3-2.

D.2.2   Guy dampers

D.2.2.1 General

(1)   To suppress the possible vibrations that can occur in guys under wind one of the following procedures should be followed:

a) Dampers should be mounted on guys in all cases where the initial tension is greater than 10% of the rated breaking strength of the guy.

b) Where guy dampers are not fitted the guys should be carefully observed during the first years of service to ensure that excessive frequency and/or amplitude of oscillations are not occurring. Otherwise dampers as described in (a) should be fitted.

   NOTE: For vibrations see Annex B.

D.2.2.2 Dampers to reduce vortex excitation

(1)   Appropriate dampers should be installed in all cases where unacceptable vortex-excited vibrations are predicted or have been observed. Dampers should conform to appropriate technical specifications. A frequency band of vibration should be specified.

D.2.2.3 Dampers to prevent galloping (including rain/wind induced vibrations)

(1)   Partial control of galloping and rain/wind induced vibrations may be obtained by the attachment of a rope from guy to guy, connecting the points of maximum amplitude of two or more guys. The effect of this under high wind conditions should be taken into account in the design of the connections to the guy.

> **NOTE:** Hanging chains may also be used to provide partial control of galloping, if the chains will operate over the relevant frequency range.

D.3   Insulators

(1)   Insulators should be selected dependent on electrical and mechanical requirements.

(2)   The minimum ultimate strength should be taken from relevant technical specifications.

(3)   Each guy insulator fitting should be designed such that even if an insulator suffers electrical failure the stability of the mast is still ensured. This may be achieved, for example by the use of fail safe insulators or insulators in parallel.

(4)   Arcing arrangements should be made such that arcing will not occur along the surface of the insulating materials adjacent to the steel fitting.

(5)   Where insulators are used at the base of the mast, jacking facilities should be provided to enable replacement of units.

(6)   Mechanical loading and unloading for ceramic insulating material (during mechanical tests and/or during construction) should be carried out in accordance with the relevant technical specifications.

> **NOTE 1:** The National Annex may give further information. In the absence of other data loading and unloading should be undertaken at a rate of approximately 5% of the expected load in steps of approximately 1 minute, such that any loading or unloading will take not less than 20 minutes.

> **NOTE 2:** For electrical properties see National Annex.

D.4   Ancillaries and other items

D.4.1   Ladders, platforms, etc.

(1)   Ladders, platforms, safety rails and other ancillaries should comply with the relevant specifications.

> **NOTE:** The National Annex may give further information.

D.4.2   Lightning protection

(1)   Towers, masts and guys should be effectively earthed for protection against lightning. This may be achieved by a metallic tape ring around the base connected to metallic plates and rods embedded in the soil. Guy anchors should be similarly protected.

(2)   The earthing system should be completed before erection of the steelwork, and connections should be made to the stay earthing system as erection work proceeds.

(3)   Provided that all the structural joints are electrically continuous, no further bonding need be incorporated.

NOTE: The National Annex may give further information.

D.4.3 Aircraft warning

(1) Structures that constitute a hazard to aerial navigation should be marked.

NOTE: The National Annex may give further information.

D.4.4 Protection against vandalism

(1) Suitable protective measures should be installed to restrict access by unauthorized persons.

NOTE: The National Annex may give further information.

Annex E [informative] – Guy rupture

E.1   Introduction

(1)   The precise analysis of a guyed mast for the dynamic actions caused by a sudden rupture of a guy is very complicated, because several of the different factors influencing the behaviour of the mast immediately after failure are uncertain; for instance the character of the rupture, the damping of the guys and the mast, the vibration of the guys and the mast, etc. Accordingly, the simplified analytical model given in E.2 may be adopted. A conservative procedure is given in E.3.

E.2   Simplified analytical model

(1)   For the simplified analysis of a guyed mast due to the rupture of a guy, the dynamic forces should be assumed to be equivalent to a static force acting on the mast at the level of the set of guys where rupture has assumed to have occurred.

(2)   For the calculation of this static equivalent force $F_{h,dyn,Sd}$ described below, it is assumed that:

–   the rupture is a simple cut through the guy;

–   the elastic energy stored in guy 1 (see Figure E.1) before the rupture occurs is neglected;

–   damping is not taken into account;

–   the wind loading when calculating the equivalent force is neglected.

1 Guy 1
2 Guy 2
3 Guy 3
4 Deflection

Elevation   Section at top level
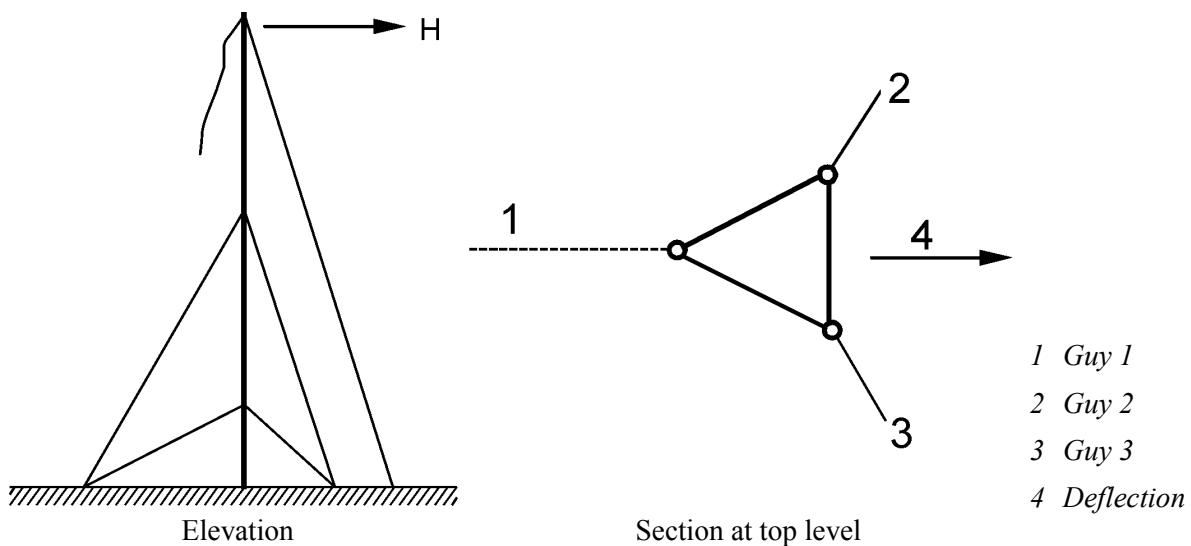
Figure E.1 Guy rupture

(3)   For a given deflection u guys 2 and 3 act on the mast shaft with a force $F_{h,Sd}$. The relation is shown in Figure E.2 as curve 1. It will be seen that $F_{h,Sd}$ decreases with increasing deflection owing to slackening of the guys.

(4)   For the mast system, except for the set of guys at the considered level, the relation between an external horizontal force and the deflection of the node can be shown as well. In Figure E.2 this relation is shown in curve 2. Where the two curves 1 and 2 intersect, the two forces are equal, i.e. there is static equilibrium. The force acting on the joint is $F_{h,stat,Sd}$.

(5)   At the moment that rupture occurs, energy is stored in guys 2 and 3. When the mast starts deflecting, this energy will partially be transformed into kinetic energy.

(6)   At the maximum deflection, the kinetic energy will be zero, because the energy lost in guys 2 and 3
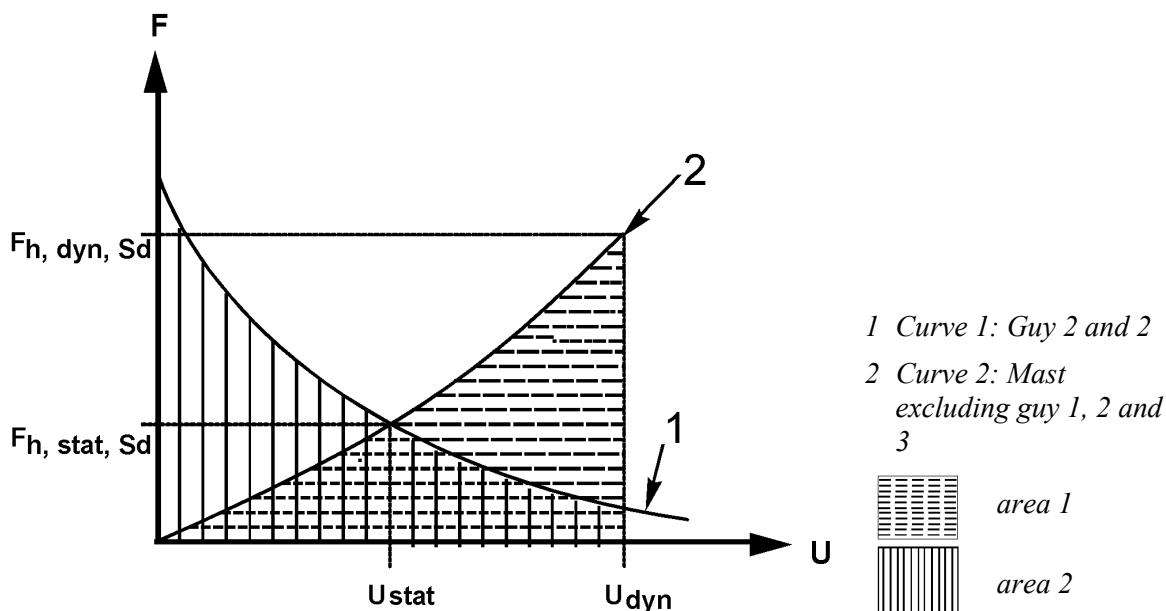
has been transferred to the mast as elastic strain energy in the shaft and the guys. Damping has not been taken into consideration.

(7)   The energy lost in guys 2 and 3 should be assumed to be equal to the area A2 below curve 1 in Figure E.2.

(8)   The deflection resulting in the two areas A1 and A2 being equal, should be taken as the dynamic deflection u_{dyn}.

(9)   The dynamic force $F_{h,dyn,Sd}$ corresponds to this dynamic deflection. The impact factor Φ may be determined using:

$$\Phi = \frac{F_{h,dyn,Sd}}{F_{h,stat,Sd}}$$ ... (E.1)

1 Curve 1: Guy 2 and 2

2 Curve 2: Mast excluding guy 1, 2 and 3

area 1

area 2

Figure E.2 Force-deflection diagram

(10)   The above procedure for the analysis of a mast just after a possible guy rupture has occurred applies to a mast guyed in 3 directions. For masts guyed in 4 (or more) directions similar procedures based on the same principles should be adopted.

(11)   If agreed between the designer, the client and the competent authority the above dynamic force arising from the rupture should not be combined with climatic loads.

E.3   Conservative procedure

(1)   The dynamic forces in the mast column and the guys caused by a cable rupture can be conservatively estimated using the following static calculations.

(2)   The horizontal component of the guy force acting in the guy before the rupture should be used as an additional force acting on the mast without the broken guy.

NOTE: In the absence of other climatic loads this corresponds to the initial tension.

(3)   The resulting guy forces should be increased by the factor 1.3 in the case of masts with 2 stay levels or if the rupture of a top guy is considered.

E.4   Analysis after a guy rupture

(1)   In addition to the procedures set out in E.2 or E.3 above, immediately after the rupture of a guy the mast should be able to withstand wind loads for a short period until temporary guying can be arranged.

(2)   If no other requirements are given, the mast without the ruptured guy should be able to withstand a reduced wind load, acting as a static load, and without patch wind loading. The reduced wind loading should be taken as 50% of the characteristic mean wind loading, acting in the most adverse wind direction.

Annex F [normative] – Fabrication and erection

F.1   General

(1)   Towers and masts should be fabricated and erected according to EN 1090.

F.2   Bolted connections

(1)   All bolt assemblies on towers and masts should be provided with suitable measures to avoid any loosening of nuts in service.

(2)   Bolt holes in members under permanent tension (e.g. cross arms) should be drilled.

(3)   Fitted or friction grip bolts, or closer tolerances on bolt holes than those given in EN 1993-1-1 may be used where displacements are critical (see 4.2.2).

F.3   Welded connections

(1)   The quality of welds assumed in selecting the appropriate fatigue class of a structural detail, see 9.3, should be specified on the drawings for the fabrication of the structure.

F.4   Tolerances

F.4.1   General

(1)   The normal tolerances given in EN 1090, Part 2 should be satisfied in fabrication.

(2)   Special tolerances should be used where normal tolerances could lead to inaccuracies of the complete structure that might cause an unacceptable reduction of its safety or serviceability.

F.4.2   Erection tolerances

F.4.2.1   Lattice towers

(1)   The maximum displacement of the tower top should be specified.

   NOTE: The National Annex may give further information. A maximum displacement of the tower top not more than $^1/_{500}$ of the height of the tower is recommended.

(2)   Final plumbing should be done in calm conditions taking due account of any temperature effects.

F.4.2.2   Guyed masts

(1)   The sensitivity to the structure to varying wind speeds for final plumbing and guy tensioning should be determined in design.

   NOTE: Generally if such operations are to be undertaken in wind speed in excess of 5m/s calculations will be required to compensate for the effects of wind, taking due account of any temperature effects.

(2)   Final plumbing and tensioning of guys should normally proceed from the lowest guy level upward.

   NOTE: The National Annex may give limits for the tolerances. The following values are recommended:

a) The final position of the centre line of the mast should all lie within a vertical cone with its apex at the mast base and with a radius of $^1/_{1500}$ of the height above the mast base. This does not apply to halyards or aerial array wires.

b) The resultant horizontal component of the initial guy tensions of all the guys at a given level should not exceed 5% of the average horizontal component of the initial guy tension for that level. The initial tension in any individual guy at a given level should in no case vary more than 10% from the design value, see EN 1993-1-11.

c) Maximum initial deflection of the mast column between two guy levels, where L is the distance between the guy levels in question, should be L/1000.

d) After erection the tolerance on the alignment of 3 consecutive guy connections on the shaft is limited to $(L_1 + L_2)/2000$, where L_1 and L_2 are the lengths of the two consecutive spans of the shaft.

F.4.3   Tensioning constraints

(1)   After erection, the guys should be tensioned in accordance with the design calculations, taking due account of the actual temperature on the site, see EN 1993-1-11.

(2)   In order to minimise the possibility of guy vibrations still air tensions should be selected such that for each guy the tension is less than 10% of its breaking load.

> **NOTE 1:** For small masts this figure may be exceeded.

> **NOTE 2:** Low still air tensions can give rise to galloping of guys.

F.5   Prestretching of guys

(1)   In order to ensure that the rope is in a truly elastic condition guys should be prestretched preferably prior to terminating. This may be done at the supplier's works or, if suitable facilities exist, at the erection site, see EN 1993-1-11.

> **NOTE:** The need for prestretching is dependent on the planned programme for retensioning, the type and size of the rope used and the sensitivity to deflections.

(2)   Prestretching should be carried out by loading the guy cyclically between 10% and 50% of its breaking load. The number of cycles should not be less than ten. This process should not be carried out by passing the loaded guy around a sheave wheel.

Annex G [informative] – Buckling of components of masts and towers

G.1 Buckling resistance of compression members

(1)   The design buckling resistance of a compression member in a lattice tower or mast should be determined according to EN 1993-1-1 as:

$$N_{b.Rd} = \frac{\chi A f_y}{\gamma_{M1}} \text{ for class 1,2 and 3 cross section} \qquad \text{... (G.1a)}$$

$$N_{b.Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \text{ for class 4 cross sections} \qquad \text{... (G.1b)}$$

where χ is the reduction factor for the relevant buckling mode defined in 6.3.1.2 of EN 1993-1-1.

(2)   For constant axial compression in members of constant cross section, the reduction factor χ should be determined with the effective slenderness ratio $\overline{\lambda}_{eff}$ instead of $\overline{\lambda}$. The effective slenderness ratio $\overline{\lambda}_{eff}$ is defined as:

$$\overline{\lambda}_{eff} = k\overline{\lambda} \qquad \text{... (G.2)}$$

.   where

k is the effective slenderness factor obtained from G.2 and

λ is the slenderness for the relevant buckling mode see Annex H.

NOTE: The effective slenderness takes into account the support conditions of the compression member.

(3)   For single angle members which are not connected rigidly at both ends (at least with two bolts, if bolted), the design buckling resistance defined in G.1(1) should be reduced by a reduction factor η.

NOTE: The reduction factor η may be defined in the National Annex. The following values are recommended:

η = 0,8 for single angle members connected by one bolt at each end;

η = 0,9 for single angle members connected by one bolt at one end and continuous or rigidly connected at the other end.

G.2 Effective slenderness factor k

(1)   In order to calculate the appropriate generalised slenderness of the member, the effective slenderness factor k may be determined according to the structural configurations.

(a) *Leg members*

   k should be obtained from table G.1.

(b) *Diagonal bracing members*
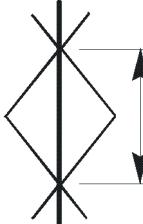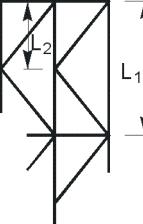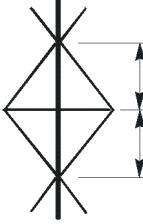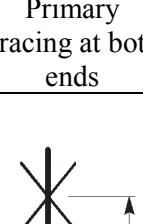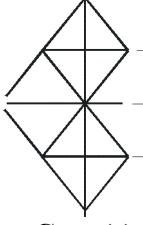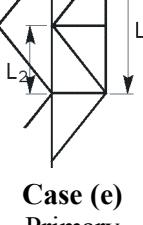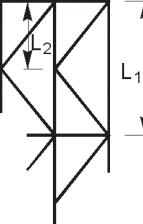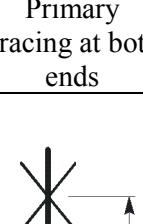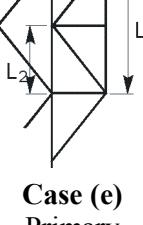
   k should be determined taking account of both the bracing pattern (see Figure H.1) and the connections of the bracing to the legs. In the absence of more accurate information values of k should be obtained from table G.2.

(c) *Horizontal bracing members*

   In the case of horizontal members of K bracing without plan bracing (see H.3.10) that have compression in one half of their length and tension in the other, the effective slenderness factor k for buckling

transverse to the frame determined from table G.2, should be multiplied by the factor k_l given in table G.3 depending on the ratio of the tension load, N_t, to the compression load N_c.

Table G.1 Effective slenderness factor k for leg members

Symmetrical bracing			Unsymmetrical bracing			
Section	∟ (3)	┼ ● ○	Section	∟ (3)		┼ ● ○
Axis	v - v	y - y	Axis	v - v	y - y	y - y
Case (a) Primary bracing at both ends	$0,8 + \dfrac{\overline{\lambda}}{10}$ but $\geq 0,9$ and $\leq 1,0$	$1,0^{(1)}$	discontinuous top end with horizontals **Case (d)** Primary bracing at both ends	$1,2\left(0,8 + \dfrac{\overline{\lambda}}{10}\right)$ but $\geq 1,08$ and $\leq 1,2$ on $L_2^{(2)}$	$1,2\left(0,8 + \dfrac{\overline{\lambda}}{10}\right)$ but $\geq 1,08$ and $\leq 1,2$ on L_1	$1,0$ on $L_1^{(1)}$
asymmetric symmetric **Case (b)** Primary bracing at one end and secondary bracing at the other	$0,8 + \dfrac{\overline{\lambda}}{10}$ but $\geq 0,9$ and $\leq 1,0$	$1,0^{(1)}$	**Case (e)** Primary bracing at both ends	$0,8 + \dfrac{\overline{\lambda}}{10}$ but $\geq 0,9$ and $\leq 1,0$ on $L_2^{(2)}$	$0,8 + \dfrac{\overline{\lambda}}{10}$ but $\geq 0,9$ and $\leq 1,0$ on L_1	$1,0$ on $L_1^{(1)}$
Case (c) Secondary bending at both ends	$0,8 + \dfrac{\overline{\lambda}}{10}$ but $\geq 0,9$ and $\leq 1,0$	$1,0$				

NOTE 1: A reduction factor may be justified by analysis.
NOTE 2: Only critical if very unequal angle section is used.
NOTE 3: The above values only apply to 90° angles.

Table G.2 Effective slenderness factor k for bracing members
(a) Single and double bolted angles
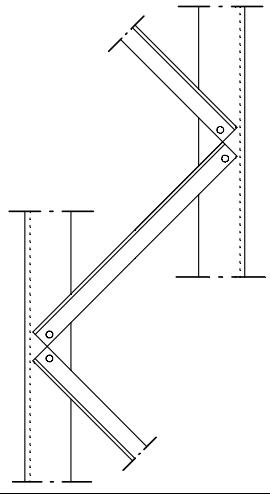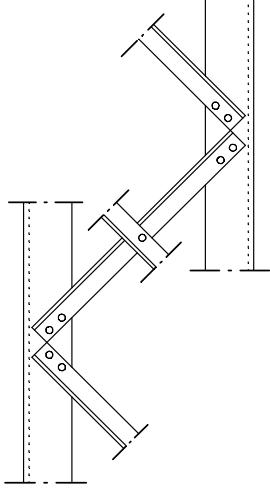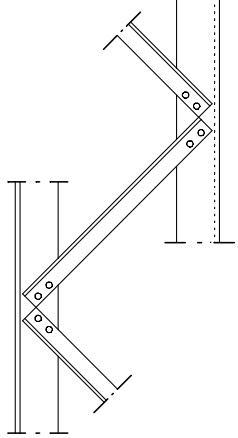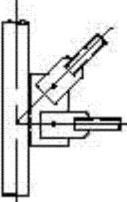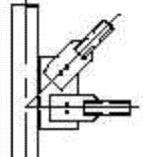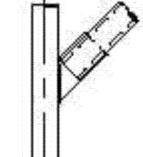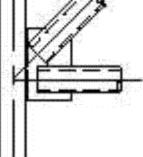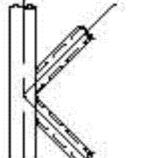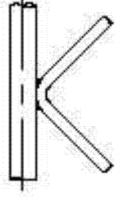
Type of restraint	Examples	Axis	k
Discontinuous both end		v-v	$0,7 + \dfrac{0,35}{\overline{\overline{\lambda}}_v}$
		y-y	$0,7 + \dfrac{0,58}{\overline{\overline{\lambda}}_y}$
		z-z	$0,7 + \dfrac{0,58}{\overline{\overline{\lambda}}_z}$
Continuous one end		v-v	$0,7 + \dfrac{0,35}{\overline{\overline{\lambda}}_v}$
		y-y	$0,7 + \dfrac{0,50}{\overline{\overline{\lambda}}_y}$
		z-z	$0,7 + \dfrac{0,50}{\overline{\overline{\lambda}}_z}$
Continuous both ends		v-v	$0,7 + \dfrac{0,35}{\overline{\overline{\lambda}}_v}$
		y-y	$0,7 + \dfrac{0,50}{\overline{\overline{\lambda}}_y}$
		z-z	$0,7 + \dfrac{0,50}{\overline{\overline{\lambda}}_z}$

Table G.2 Effective slenderness factor k for bracing members
(b) Tubes and rods

	Type	Axis	K [3][5]
tube or rod legs	single bolted tube	in plane	0,95 [2]
		out of plane	0,95 [2]
	double bolted tube / welded tubes with end plates	in plane	0,85
		out of plane	0,95 [2]
	welded tubes [1] and rods with welded gussets	in plane	0,70
		out of plane	0,85
	directly welded tubes and rods	in plane	0,70
		out of plane	0,70
	welded bent rods	in plane	0,85
		out of plane	0,85

NOTE 1: Double preloaded bolts may qualify for this condition subject to analysis.

NOTE 2: Reduction for actual length only.

NOTE 3: Where ends are not the same, an average "K" value should be used.

NOTE 4: Above details are shown for illustrative purposes only and may not reflect practical design aspects.

NOTE 5: Above values are for bracing members with the same connection type at each end. For members with intermediate secondary bracing "K" factors may increase and upper values of 1,0 should be used unless justified by tests.

Table G.3 Modification factor (k_1) for horizontal of K brace without plan bracing

Ratio $\dfrac{N_t}{N_c}$	Modification factor, k_1
0,0	0,73
0,2	0,67
0,4	0,62
0,6	0,57
0,8	0,53
1,0	0,50
A value of 1,0 applies when the ratio $\dfrac{N_t}{N_c}$ is negative (i.e. when both members are in compression).	

Annex H –[informative] Buckling length and slenderness of members

H.1 General

(1) This annex gives information about the determination of buckling length and slenderness of members in masts and towers.

H.2 Leg members

(1) The slenderness for leg members should generally be not more than 120.

(2) For single angles, tubular sections or solid rounds used for leg sections with axial compression braced symmetrically in two normal planes, or planes 60° apart in the case of triangular structures, the slenderness should be determined from the system length between nodes.

(3) Where bracing is staggered in two normal planes or planes 60° apart in the case of triangular structures, the system length should be taken as the length between nodes. The slenderness for the case shown in table G.1, case (d) should be determined from equation (H.1a) or (H.1b) as appropriate. The slenderness should be taken as:

$$\lambda = \frac{L_1}{i_{yy}} \text{ or } \lambda = \frac{L_2}{i_{vv}} \text{ for angles} \qquad \text{... (H.1a)}$$

$$\lambda = \frac{L_1}{i_{yy}} \text{ for tubes} \qquad \text{... (H.1b)}$$

NOTE: The value $\lambda = \dfrac{L_2}{i_{vv}}$ may be conservative in relation to a more refined analysis taking account of realistic end conditions.

(4) Built-up members for legs may be formed with two angles in cruciform section or of two angles back to back.

(5) Built-up members consisting of two angles back to back (forming a T) may be separated by a small distance and connected at intervals by spacers and stitch bolts. They should be checked for buckling about both rectangular axes according to 6.4.4 of EN 1993-1-1.

(6) Stitch bolts should not be assumed to provide full composite action where the gap between the angles exceeds 1,5 t, and the properties should be calculated assuming a gap equal to the true figure or 1,5 t, whichever is the lesser where t is the thickness of the angle. If batten plates are used in addition to stitch bolts the properties corresponding to the full gap should be taken. See 6.4.4 of EN 1993-1-1.

(7) Battens should prevent relative sliding of the two angles; if bolted connections of categories A and B are used, see 3.4 of EN 1993-1-8, the bolt hole diameter should be reduced.

NOTE 1: The rules (5) to (7) also apply to built-up members in legs.

NOTE 2: The National Annex may give further information.

H.3   Bracing members

H.3.1 General

(1)   The following rules should be used for the typical primary bracing patterns shown in Figure H.1. Secondary bracings may be used to subdivide the primary bracing or main leg members as shown, for example, in Figures H.1 (IA, IIA, IIIA, IVA) and H.2.

(2)   The slenderness λ for bracing members should be taken as:

$$\lambda = \frac{L_{di}}{i_{vv}} \text{ for angles} \qquad\qquad ... \text{(H.2a)}$$

$$\lambda = \frac{L_{di}}{i_{yy}} \text{ for tubes} \qquad\qquad ... \text{(H.2b)}$$

where L_{di} is specified in Figure H.1

> **NOTE:** The value $\lambda = \dfrac{L_{di}}{i_{vv}}$ may be conservative in relation to a more refined analysis taking account of realistic end conditions.

(3)   The slenderness λ for primary bracing members should generally be not more than 180 and for secondary bracing not more than 250. For multiple lattice bracing (Figure H.1(V)) the overall slenderness should generally be not more than 350.

> **NOTE:** The use of high slenderness ratios can lead to the possibility of individual members vibrating and can make them vulnerable to damage due to bending from local loads.

H.3.2 Single lattice

(1)   A single lattice may be used where the loads are light and the lengths relatively short, as for instance near the top of towers or in light masts (see Figure H.1(I)). The slenderness may be taken as:

Typical primary spacing patterns					
parallel or tapering			usually tapering		usually parallel
					Tension member
I	II	III	IV Discontinuous bracing with continuous horizontal intersections	V	VI
Single lattice	Cross bracing	K-bracing		Multiple lattice bracing	Tension bracing
$L_{di} = L_d$	$L_{di} = L_{d2}$	$L_{di} = L_{d2}$	$L_{di} = L_{d2}$		
Typical secondary bracing patterns (see also Figure H.2)				NOTE: The tension members in pattern VI are designed to carry the total shear in tension, e.g.	
IA	IIA Cross bracing	IIIA	IVA Cross bracing with secondary members		
	$L_{di} = L_{d1}$	$L_{di} = L_{d1}$	$L_{di} = L_{d1}$		

Figure H.1 Typical bracing patterns

H.3.3 Cross bracing

(1)   Provided that the load is equally split into tension and compression, the members are connected where they cross, and provided also that both members are continuous (see Figure H.1(II)), the centre of the cross may be considered as a point of restraint both transverse to and in the plane of the bracing and the critical system length becomes L_{d2} on the minor axis.

(2)   Where the load is not equally split into tension and compression and provided that both members are continuous, the compression members should be checked in the same way for the largest compressive force. In addition, it should be checked that the sum of the buckling resistances of both members in compression is at least equal to the algebraic sum of the axial forces in the two members. For the calculation of the buckling resistances, the system length should be taken as L_d and the radius of gyration as that about the rectangular axis parallel to the plane of the bracing. The slenderness may be taken as:

$$\lambda = \frac{L_d}{i_{yy}} \text{ or } \frac{L_d}{i_{zz}} \qquad \text{for angles} \qquad \qquad \text{... (H.3a)}$$

$$\lambda = \frac{L_d}{i_{yy}} \qquad \qquad \text{for tubes or solid rounds} \qquad \qquad \text{... (H.3b)}$$

NOTE: Where either member is not continuous, the centre of the connection may only be considered as a restraint in the transverse direction if the detailing of the centre connection is such that the effective lateral stiffness of both members is maintained through the connection and the longitudinal axial stiffness is similar in both members.

H.3.4 Tension bracing

(1)   Each diagonal member of a pair of tension bracing members and the horizontals should be capable of carrying the full bracing shear load (see Figure H.1 (VI)).

NOTE: Tension systems are very sensitive to methods of erection and to modifications or relative movements. Detailing to give an initial tension within the bracing and to provide mutual support at the central cross will be required to minimise deflection.

H.3.5 Cross bracing with secondary members

(1)   Where secondary members are inserted to stabilize the legs (see Figure H.1(IIA and IVA) and Figure H.2(a)), reducing the buckling length on the minimum axis to L_{d1}.

(2)   Buckling should also be checked over length L_{d2} on the rectangular axis for buckling transverse to the bracing and then over length L_d for the algebraic sum of the axial forces, see H.3.3.

H.3.6 Discontinuous cross bracing with continuous horizontal at centre intersection

(1)   The horizontal member should be sufficiently stiff in the transverse direction to provide restraints for the load cases where the compression in one member exceeds the tension in the other or where both members are in compression, see Figure H.1 (IV).

(2)   This criterion may be satisfied by ensuring that the horizontal member withstands (as a compression member over its full length on the rectangular axis) the algebraic sum of the axial force in the two members of the cross-brace, resolved in the horizontal direction.

NOTE: Additional allowance can be necessary for the bending stresses induced in the edge members by local loads transverse to the frame, such as wind.

H.3.7 Cross bracing with diagonal corner members

(1)   In some patterns of cross bracing a corner member may be inserted to reduce the buckling length transverse to the plane of bracing (see Figure H.2(b)). A similar procedure to that used for H.3.3 may be used to determine whether this will provide a satisfactory restraint.

(2)   In this case five buckling checks should be carried out as follows:

–   Buckling of member against the maximum load over length L_{d1} on the minimum axis;

–   Buckling of member against the maximum load over length L_{d2} on the transverse rectangular axis;

– Buckling of two members in cross brace against the algebraic sum of loads in cross brace over the length L_{d3} on the transverse axis;

– Buckling of two members (one in each of two adjacent faces) against the algebraic sum of the loads in the two members connected by the diagonal brace over length L_{d4} on the transverse axis.

> **NOTE:** For this case the total resistance should be calculated as the sum of the buckling resistances of both members in compression (see H.3.3(2)).

– Buckling of four members (each member of cross brace in two adjacent faces) against the algebraic sum of loads in all four members over length L_d on the transverse axis.

H.3.8 Diagonal members of K bracing

(1) In the absence of any secondary members (see Figure H.1(III)) the critical system length may be taken as L_{d2} on the minor axis.

(2) Where secondary bracing in the faces is provided but no hip bracing (see Figure H.1(IIIA) the critical system length should be taken as L_{d2} on the appropriate rectangular axis. Thus the slenderness should be taken as:

$$\lambda = \frac{L_{d2}}{i_{yy}} \text{ or } \frac{L_{d2}}{i_{zz}} \qquad\qquad \text{... (H.4)}$$

(3) Where secondary bracing and triangulated hip bracing is provided (see Figure H.2(c)), then the appropriate system length between such hip members L_{d4} should be used for checking buckling transverse to the face bracing on the appropriate rectangular axis. Thus the slenderness may be taken as:

$$\lambda = \frac{L_{d4}}{i_{yy}} \text{ or } \frac{L_{d4}}{i_{zz}} \text{ for all types of section} \qquad\qquad \text{... (H.5)}$$

H.3.9 Horizontal face members with horizontal plan bracing

(1) Where the length of the horizontal face members becomes large, plan bracing may be introduced to provide transverse stability.

(2) The system length of the horizontal member for buckling should be taken as the distance between intersection points in the plan bracing for buckling transverse to the frame, and the distance between supports in plan for buckling in the plane of the frame.

(3) Care should be taken in the choice of the vv or rectangular axes for single angle members. The vv axis should be used unless suitable restraint by bracing is provided at or about the mid-point of the system length. In this case buckling should be checked about the vv axis over the intermediate length and about the appropriate rectangular axis over the full length between restraints on that axis.

> **NOTE:** This procedure may be conservative in relation to a more refined analysis taking account of realistic end conditions.

(4) Where the plan bracing is not fully triangulated, additional allowance should be made for the bending stresses induced in the edge members by loads, such as wind transverse to the frame, see Figure H.3.

(5) To avoid buckling, where the plan bracing is not fully triangulated:

– the horizontal plan bracing should be designed to resist a concentrated horizontal force of $p \times H$ applied at the middle of the member where p is the percentage of the maximum axial compression force, H, in the horizontal plan bracing, as given in 6.4;

– the deflection of horizontal plan bracing under this force should not exceed L/500.

(a) [IIB]

(b) [IIC]
Cross bracing with diagonal corner
members

*1 corner stay (of
limited effect if
both braces are
in compression)*

*fully
triangulated
hip bracing*

2 hip bracings

(c) [IIIB]
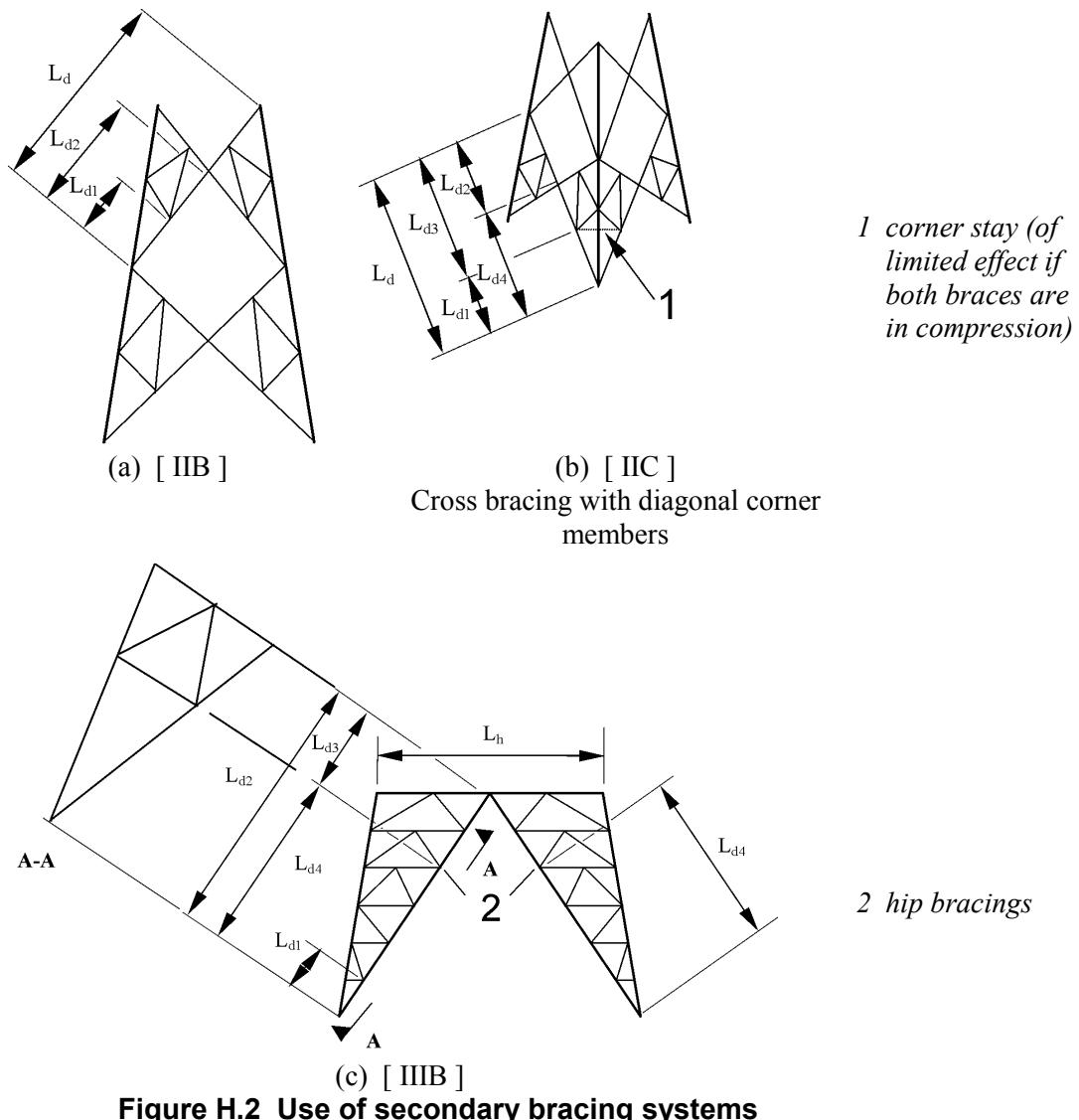
Figure H.2 Use of secondary bracing systems

| Triangulated |
|---|
|

* If there are two diagonals, they may be designed as tension members. |
| **Not fully triangulated**
(not recommended for design, unless careful attention is given to bending effects) |
| |
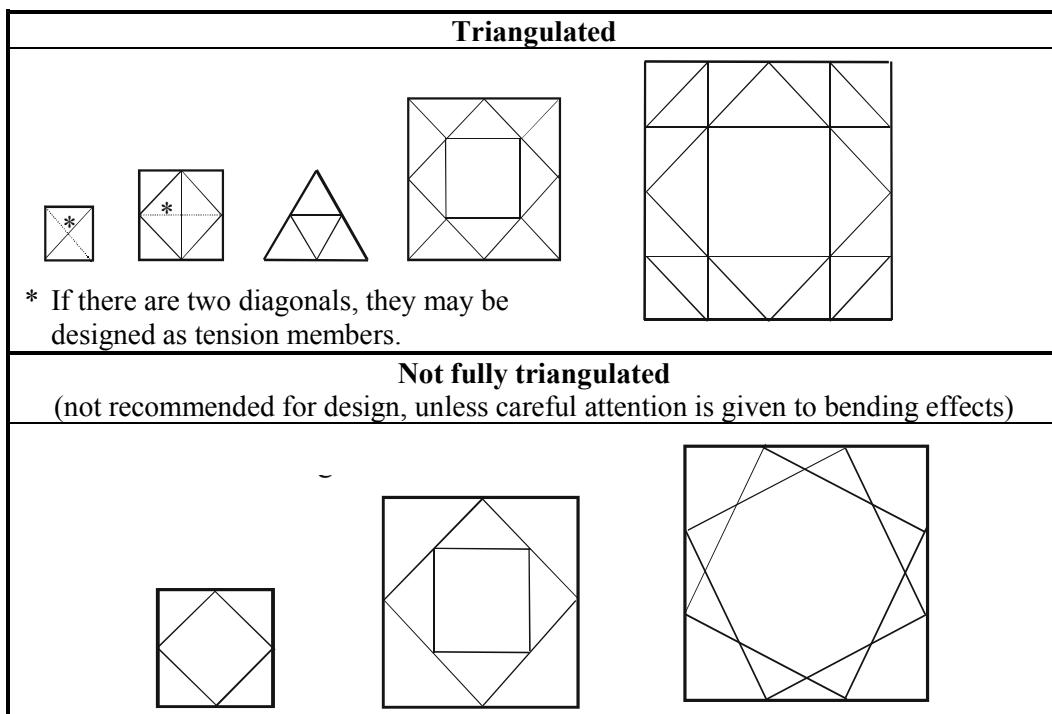
Figure H.3 Typical plan bracing

H.3.10 Horizontal members without plan bracing

(1)   For small widths of towers and for masts plan bracing may be omitted in appropriate cases with due justification.

(2)   The rectangular radius of gyration should be used for buckling transverse to the frame over length L_h (see Figure H.4(a)). However for single angle members, the radius of gyration about the vv axis should be used over length L_{h2} unless restraint by secondary bracing at intervals along the length is provided in which case the system length should be taken as L_{h1}, see Figure H.4(b).

> **NOTE:** This procedure may be conservative in relation to a more refined analysis taking account of realistic end conditions.

(3)   To avoid buckling of the horizontal member the criteria of H.3.9(5) should be satisfied.

> **NOTE:** Additional allowance may be necessary for the bending stresses induced in the edge members by local loads transverse to the frame, such as wind.

$$\lambda = L_{h2} / i_{vv} \quad \text{for angles}$$
$$\text{and } (\lambda = L_h / i_{zz})$$

$$\lambda = L_h / i_{yy} \quad \text{for tubes}$$

$$\lambda = L_{h1} / i_{vv} \quad \text{for angles}$$
$$\text{and } (\lambda = L_h / i_{zz})$$

$$\lambda = L_h / i_{yy} \quad \text{for tubes}$$
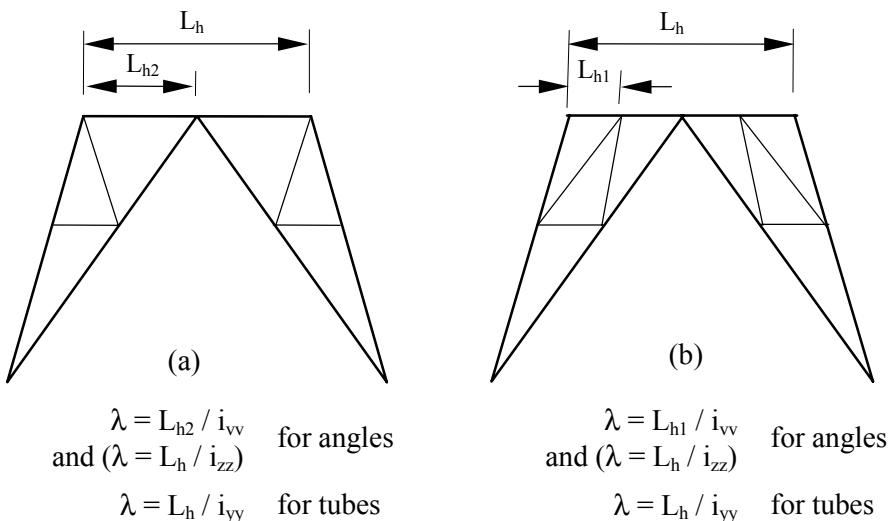
Figure H.4 K bracing horizontals without plan bracing

H.3.11 Cranked K bracing

(1)   For large tower widths, a crank or bend may be introduced into the main diagonals (see Figure H.5), which has the effect of reducing the length and size of the redundant members. As this produces high stresses in the members meeting at the bend, transverse support should be provided at the joint. Diagonals and horizontals should be designed as for K bracing, system lengths of diagonals being related to the lengths to the knee joint.

H.3.12 Portal frame

(1)   A horizontal member may be introduced at the bend to turn the panel into a portal frame, see Figure H.6. Because this leads to a lack of articulation in the K brace, special consideration should be given to the effects of foundation settlement or movement.
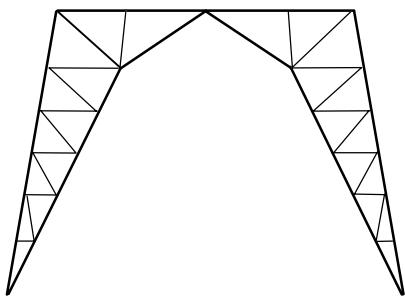

Figure H.5 Cranked K bracing
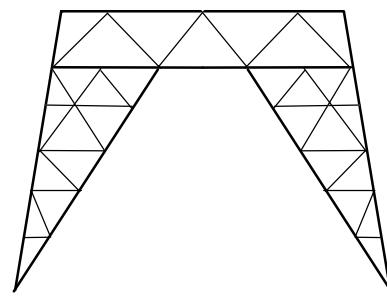
Figure H.6 Portal frame

H.3.13 Multiple lattice bracing

(1)   In a multiple lattice configuration the bracing members that are continuous and connected at all intersections should be designed as secondary members (see 6.4) on a system length from leg to leg with the appropriate radius of gyration i_{yy} or i_{zz}, see Figure H.7. For the stability of the panel the overall slenderness

$\dfrac{L}{i_{yy}}$ should be less than 350. For single angle members $\dfrac{i_{yy}}{i_{vv}}$ should be greater than 1,50 where i_{yy} is the radius of gyration about the axis parallel to the plan of the lattice.

(2)   The stability of the member A-B shown in Figure H.7 should be checked under the applied force on the critical system length L_o for the slenderness:

$$\lambda = \dfrac{L_o}{i_{vv}} \text{ for angles} \qquad\qquad\qquad \text{... (H.6a)}$$

$$\lambda = \dfrac{L_o}{i_{yy}} \text{ for tubes and solid rounds} \qquad\qquad \text{... (H.6b)}$$

NOTE: The value of $\lambda = \dfrac{L_o}{i_{vv}}$ may be conservative in relation to a more refined analysis taking account of realistic end conditions.
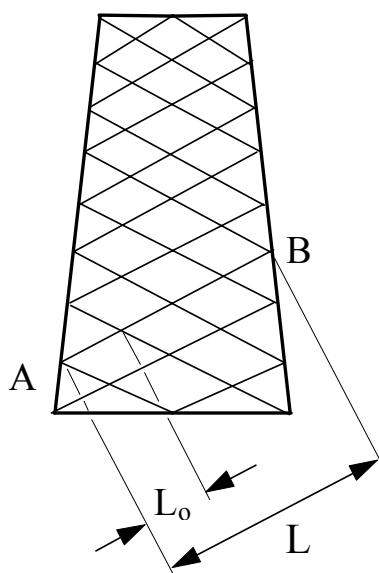

Figure H.7 Multiple lattice bracing

H.4   Secondary bracing members

(1)   In order to allow for imperfections in leg members, and for the design of secondary bracing members, a notional force should be introduced acting transverse to the leg member (or other chord if not a leg) being stabilized at the node point of the attachment of the bracing member. Depending on the slenderness of the leg member being stabilized, the value of the notional force to be used for the design of any secondary member should be obtained from (2) and (3).

(2)   The force to be applied at each node in turn in the plane of bracing, expressed as a percentage, p, of the axial force in the leg for various values of the slenderness λ of the leg may be taken as:

$$p \quad = \quad 1,41 \text{ when } \lambda < 30 \qquad\qquad\qquad \text{... (H.7a)}$$

$$p \quad = \quad \dfrac{(40+\lambda)}{50} \text{ when } 30 \le \lambda \le 135 \qquad\qquad \text{... (H.7b)}$$

$$p \quad = \quad 3,5 \text{ when } \lambda > 135 \qquad\qquad\qquad \text{... (H.7c)}$$

(3)   When there is more than one intermediate node in a panel then the secondary bracing system should be checked separately for 2,5% of the axial force in the leg shared equally between all the intermediate node points. These notional forces should be assumed to act together and in the same direction, at right angles to the leg and in the plane of the bracing system.

(4)   In both cases (2) and (3) the distribution of forces within the triangulated secondary bracing panel should be determined by linear elastic analysis.

(5)   The effects of this notional force should generally be added to the primary force as calculated from the global analysis for the design of any primary member. Exceptionally for self-supporting lattice towers of conventional configuration the notional forces need not be added to the primary forces, provided that the primary bracing is checked for the effects of the notional force, when the primary force is smaller. For guyed masts the effects of the notional force should always be added to the primary force.

(6)   Provided that it is designed for notional forces as described in (1) to (5) it may be assumed that the stiffness of the bracing system will be sufficient.

(7)   If the main member is eccentrically loaded or the angle between the main diagonal of a K brace and the leg is less than 25° then the above value of the notional force may be insufficient and a more refined value should be obtained by taking into account the eccentricity moment and secondary stresses arising from leg deformation.

(8)   Where the direction of buckling is not in the plane of the bracing, then the values given by equations 5.17 a), b) and c) should be divided by a factor of $\sqrt{2}$.

H.5   Shell structures

(1)   For the strength and stability of shell structures see EN 1993-1-6.

    NOTE: See also EN 1993-3-2.