BS 8100-3:1999

Incorporating Corrigendum No. 1

Lattice towers and masts —

Part 3: Code of practice for strength assessment of members of lattice towers and masts

ICS 91.080.10

Committees responsible for this British Standard

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British Telecommunications plc
Department of Trade and Industry (Electricity Department)
Institution of Civil Engineers
Meteorological Office
Ministry of Defence
National Transcommunications Ltd
Steel Construction Institute
UK Steel Association

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Contents

Foreword

This part of BS 8100 has been prepared under the direction of Technical Committee B/525, Building and civil engineering structures. BS 8100 is a series of standards combining codes of practice with a guide covering the loading and design of lattice towers and masts of metallic construction. It comprises the following parts:

— *Part 1: Code of practice for loading*;

— *Part 2: Guide to the background and use of Part 1: "Code of practice for loading"*;

— *Part 3: Code of practice for strength assessment of members of lattice towers and masts*;

— *Part 4: Code of practice for loading of guyed masts*.

This part of BS 8100 incorporates clauses to complement BS 8100-4.

The need for this code of practice to give guidance on the determination of the strength of lattice towers and masts arises out of the difficulty in applying consistently the general structural strength codes to these often tall, slender three dimensional lattice structures.

The document is based upon DD 133:1986, which in turn brought together the collective experience of the European transmission line tower industry, where each new design of structure was subjected to full-scale "type" testing. It was considered invaluable to have such relatively unique data built into a code as well as enabling the limit state loading rules to be accurately calibrated.

The additional clauses dealing with the special aspects of guyed masts represent the collective extensive experience of the drafting committee. It is considered that they will provide efficient solutions to the strength assessment problem, whilst avoiding some of the causes of problems which have occurred on this type of structure.

By liaison with the relevant Eurocode drafting team, the rules drafted into this code of practice have been based upon a common approach and should result in the minimum amount of variation when the National Application Document is prepared following the issue of the ENV.

This part of BS 8100 does not apply to other metallic structures. Other British Standards exist for some of those structures.

It has been assumed in the drafting of this British Standard that the execution of its provisions will be entrusted to appropriately qualified and experienced people.

As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

Annex A, annex B and annex C are informative.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i and ii, pages 1 to 26, an inside back cover and a back cover.

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Introduction

The strengths given in this part of BS 8100 are characteristic (95 % probability) values for use with BS 8100-1 and -4. The design strengths are obtained by dividing the characteristic strengths by the appropriate partial factor γ_m defined in this part.

Connection eccentricities assumed in this part of BS 8100 are those that are traditionally applied to lattice masts and towers in the UK.

1 Scope

This part of BS 8100 provides guidance on the assessment of the strength of members and connections for masts and towers of lattice construction consisting mainly of bolted, rivetted or welded steel angle or tubular or solid round sections. The recommendations given are valid for both equal and unequal angles, either rolled or cold formed, compound members fabricated from these sections, tubular members and solid rounds.

The specific components covered in this part of BS 8100 are:

a) compression members and bolted and/or welded connections which are specific to typical mast and tower configurations and which are not comprehensively covered in general strength codes;

b) mast guys, their fittings and assemblies. For such components their strength is closely related to practical considerations, and such guidance is included where relevant.

NOTE 1 Information on all other aspects of strength assessment can be obtained from BS 5950-1. Information for fatigue assessment can be obtained from BS 7608.

2 Normative references

The following normative documents contain provisions which, through reference in this text, constitute provisions of this part of this British Standard. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

BS 302 (all parts), *Stranded steel wire ropes.*

BS 443, *Specification for testing zinc coatings on steel wire and for quality requirements.*

BS 463, *Specification for sockets for steel wire ropes.*

BS 643, *Specification for white metal ingots for capping steel wire ropes.*

BS 1554, *Specification for stainless and heat-resisting steel round wire.*

BS 2763, *Specification for round carbon steel wire for wire ropes.*

BS 4429, *Specification for rigging screws and turnbuckles for general engineering, lifting purposes and pipe hanger applications.*

BS 5950-1:1990, *Structural use of steelwork in building — Part 1: Code of practice for design in simple and continuous construction: hot rolled sections.*

BS 6994, *Specification for steel shackles for lifting and general engineering purposes: grade M(4).*

BS 7035, *Code of practice for socketing of stranded steel wire ropes.*

3 Definitions and symbols

3.1 Definitions

For the purposes of this part of BS 8100 the following definitions apply.

3.1.1

leg members

members forming the main load bearing chords of the structure

3.1.2

primary bracing members

members other than legs carrying the shear force due to imposed loads on the structure

3.1.3

secondary members

members used to reduce the effective length of the main legs and sometimes that of the bracing. They are normally considered unstressed and are only loaded due to deformation of the structure (see **5.4**.)

NOTE Secondary members are sometimes referred to as redundant members.

3.1.4

system length

taken as the geometric length of a member between intersection points providing restraint in the relevant plane

3.1.5 mast guy assemblies

3.1.5.1

guy

tension only member which provides horizontal support to the mast column at discrete levels. The lower end of the guy assembly is anchored to the ground and normally incorporates a means of adjusting the tension in the guy

NOTE The terms “stay” and “guy” are interchangeable but in this British Standard the word “guy” has been used throughout.

3.1.5.2

wire

individual filament of steel, forming the smallest single tension component in a cable, usually circular in cross section with a diameter between 3 mm and 8 mm, with a high strength that is usually obtained by cold drawing or cold rolling

3.1.5.3

spiral strand

assembly of round wires formed helically around a centre wire in one or more symmetrical layers

NOTE Each successive layer is wound in alternate directions [see Figure 1a)].

3.1.5.4

wire rope

assembly of spiral strands wound helically around a centre core, normally in the opposite direction to the outer layer of the strands [see Figure 1b)]

3.1.5.5

cable

assembly of one or more strands or ropes of the types described in **3.1.5.3** and **3.1.5.4**

NOTE The strands may be bound in tight contact or kept apart by spacers. [See Figure 1c).]

3.1.5.6

locked coil

strand comprising a central core of round wires, spirally bound with layers of shaped wires

NOTE The inner layers of shaped wires are sometimes of a trapezoidal cross section, but more usually all layers are of Z section wires which overlap and provide a relatively watertight envelope. Each successive layer is wound in alternate directions. [See Figure 1d).]

3.1.5.7

parallel wire strand

assembly of wires laid side by side in parallel and either bound tightly together or with the spaces between the wires held constant by spacer elements

3.1.5.8

core

central foundation for the strands, providing support and keeping them in their proper position throughout the life of the rope

NOTE Fibre cores impregnated with grease are used to provide internal lubrication of moving ropes. However, only steel wire cores should be used for mast guys.

3.1.5.9

termination

device fixed to the ends of a cable, strand or wire to enable the loads to be transferred between it and the mast column or the steelwork attached to the guy foundation

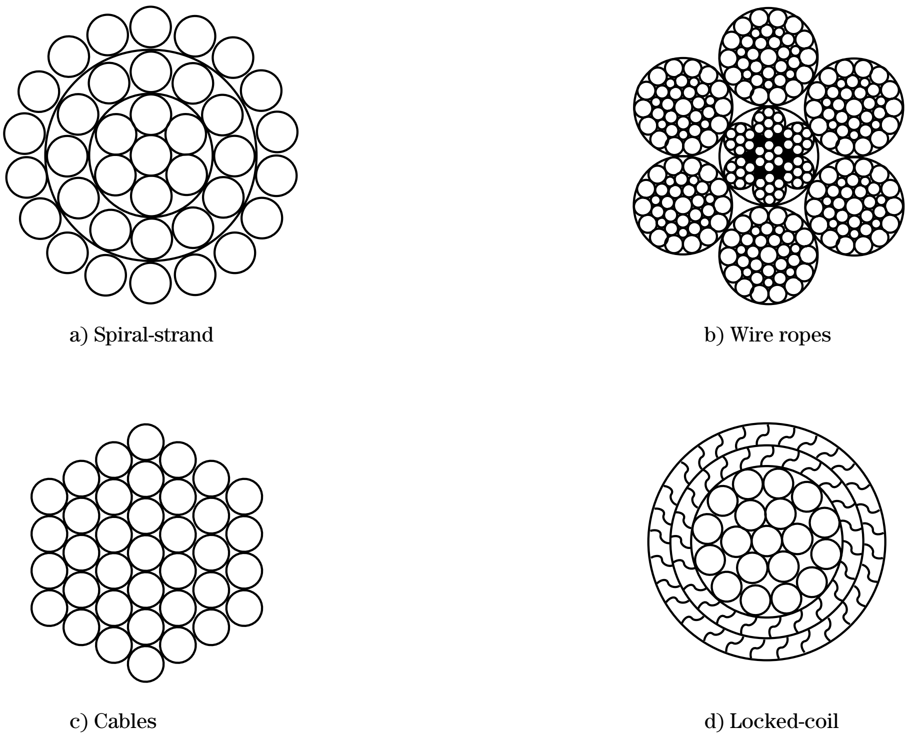


Figure 1 — Typical cable construction

3.2 Symbols

For the purposes of this part of BS 8100 the following symbols apply.

NOTE They are based on those given in BS 8100-1 and -4 but it has not been possible to follow those conventions in all cases.

A	cross-sectional area of the member
A_s	cross-sectional area of bolt in shear plane (shank or threaded section as relevant) (in mm^2)
B	leg length of angle, i.e. breadth
C	length of individual angle between stitch bolts
D	bolt, solid round or tube diameter; depth of section
E	modulus of elasticity of steel
F	leg load
j	reduction factor
K	effective slenderness factor NOTE $K\Lambda$ is effective slenderness and $K\lambda$ is effective slenderness ratio.
L	length
L_d	system length of diagonal member
L_h	system length of horizontal member
N	the design buckling resistance of a member
P_1	compressive force in horizontal of K brace
P_2	tensile force in horizontal of K brace
Q_u	ultimate resistance of bolt
r_{vv}	radius of gyration about axis vv
r_{xx}	radius of gyration about axis xx
r_{yy}	radius of gyration about axis yy
t	thickness of angle leg, thickness of tube wall
x	distance from centre line of bolt to end of member
y	distance from centre line of bolt to edge of member
z	spacing between bolts
α	imperfection factor
β	ratio of effective to gross area
λ	slenderness ratio, i.e. system length divided by radius of gyration
Λ	relative slenderness (λ/λ_1)
Λ_{eff}	effective slenderness (non- dimensional parameter incorporating K)
σ_b	specified minimum yield strength of bolt steel
σ_r	reference stress of member
σ_t	specified minimum tensile strength of bolt
σ_y	specified minimum yield strength of steel member
σ_u	specified minimum tensile strength of steel member
χ	reduction factor for relevant buckling mode

4 Axes of members

For the purposes of this part of BS 8100 the axes of members are shown in Figure 2.

5 Structural configurations, system length and effective slenderness

5.1 General

There are a number of different configurations that are commonly used in lattice towers and masts. Each requires separate consideration. Some popular arrangements are dealt with in **5.2** and **5.3**.

In order to determine the buckling resistance of a lattice member it is necessary first to derive the appropriate geometric length of the member between intersection points providing restraint, defined as the "system" length. The relevant slenderness ratio based on the system length and appropriate radius of gyration is calculated and the effective slenderness determined appropriate to the end conditions of the member (see **5.5**).

5.2 Leg members

5.2.1 *Single members*

Single angles, tubular sections or solid rounds may be used for leg sections. The capacity of the leg will be dependent on the pattern and connections of bracings used to stabilize the leg. For legs or chords with axial compression load braced symmetrically in two normal planes or planes 60° apart, for example in the case of triangular structures, the slenderness should be determined from the system length between nodes, i.e. intersections of bracings. Effective slenderness factors are given in Table 2.

When this 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 on one face. Modified effective slenderness parameters are given for angle legs in Table 2 to allow for their torsional instability in such cases.

NOTE A maximum effective slenderness ratio of 120 for leg members is good practice.

5.2.2 *Compound members*

Compound members for legs may be built up with two angles in cruciform section and then taken as fully composite if welded continuously [see Figure 3a)]. When intermittently connected [see Figure 3b)] the possible additional deformations due to shear should be taken into account by modifying the slenderness ratio, λ, in accordance with the following:

$$\lambda^2 = \lambda_0{}^2 + \lambda_1{}^2$$

where

λ_0 is the slenderness ratio of the full member;

λ_1 is the slenderness ratio of one component angle (C/r_{vv}).

NOTE It is good practice to have $\lambda_0 \geq \lambda_1$ and $\lambda_1 \leq 50$

A method for determining the size of battens is given in annex A.

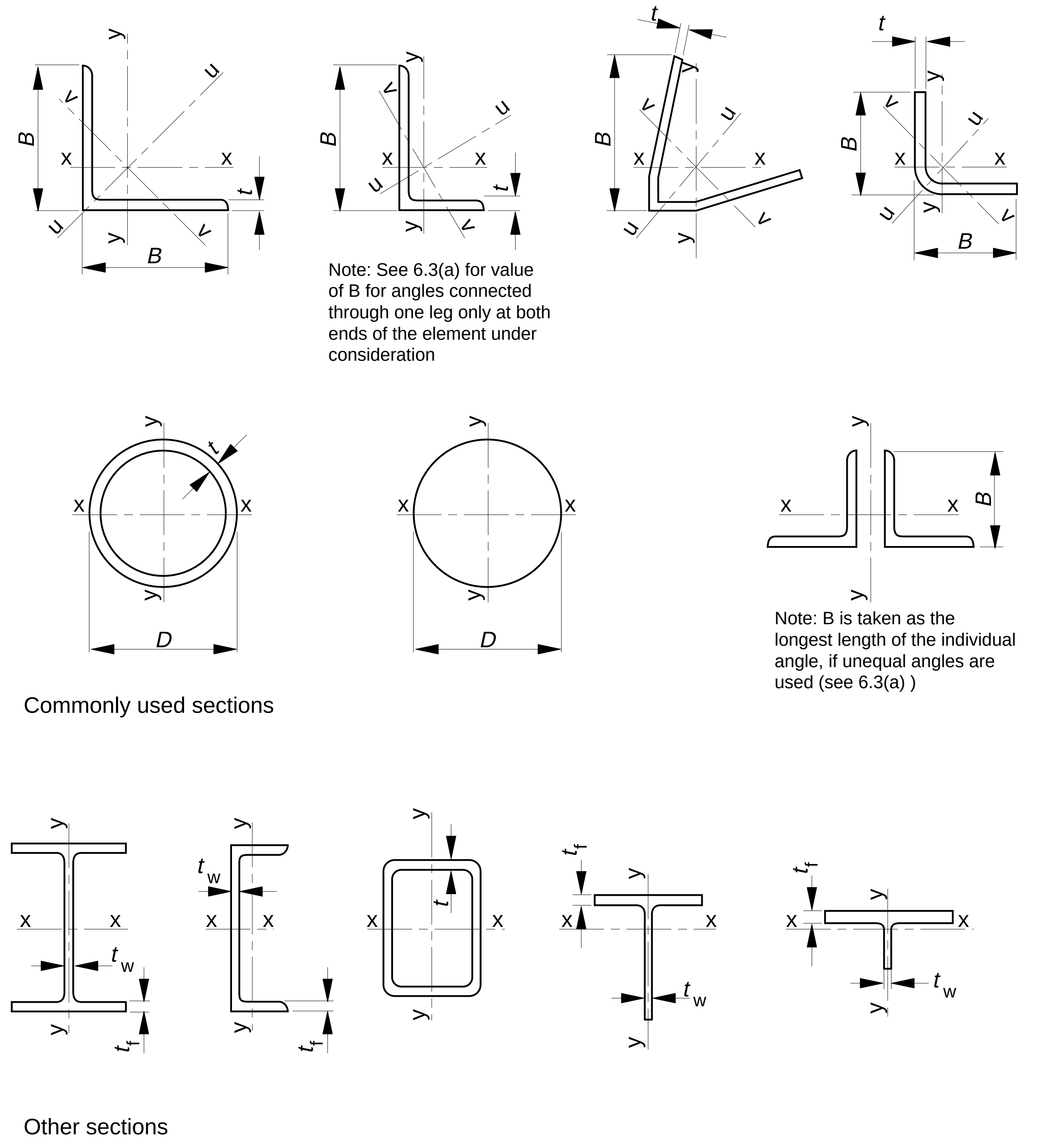


Figure 2 — Dimensions and axes of sections

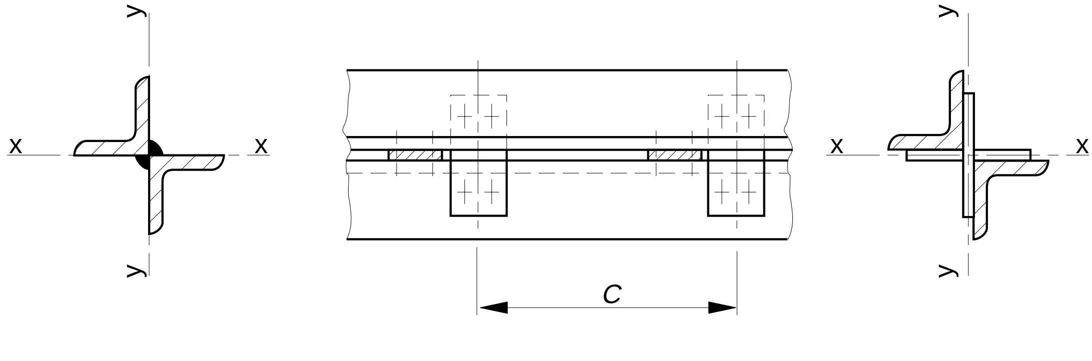


Figure 3 — Cruciform angle sections

5.3 Primary bracing members

5.3.1 *General*

Typical primary bracing patterns are shown in Figure 4.

Secondary bracings can be used to subdivide the primary bracing or main leg members as shown, for example, in Figures 4 and 5.

NOTE A maximum effectiveness slenderness ratio of 180 for primary bracing members is good practice.

5.3.2 *Single lattice*

A single lattice is commonly used where the loads are light and the length relatively short, as for instance near the top of towers or in light guyed masts [see Figure 4a) and g)]. The slenderness ratio, λ, should be taken as:

$\lambda = L_d/r_{vv}$ for angles;

$\lambda = L_d/r_{xx}$ for tubes or solid rounds.

5.3.3 *Cross bracing systems*

5.3.3.1 *Cross bracing without secondary members*

Provided that:

a) the load is equally split into tension and compression; and

b) both members are continuous [see Figure 4b)]; and

c) the members are adequately connected where they cross, then the centre of the connection may be considered as a point of restraint both transverse to and in the plane of the bracing and the critical system length becomes length L_{d2} on the minimum axis. The slenderness ratio, λ, should be taken as:

$\lambda = L_{d2}/r_{vv}$ for angles;

$\lambda = L_{d2}/r_{xx}$ for tubes.

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.

When the load is not equally split into tension and compression and provided both members are continuous, the compression members should be checked for the worst compressive load. In addition, it should be checked that the sum of the load carrying capacities of both members in compression is at least equal to the algebraic sum of the loads in the two members. For the calculations of these load carrying capacities, the system length is the whole length L_d and the radius of gyration is that about the rectangular axis parallel to the plane of the bracing. The value of the slenderness ratio, λ, should be taken as one of the following:

$\lambda = L_d/r_{xx}$ or $\lambda = L_d/r_{yy}$ for angles;

$\lambda = L_d/r_{xx}$ for tubes or solid rounds.

5.3.3.2 *Cross bracing with secondary members*

Where secondary members are inserted [see Figure 4h) and Figure 5a) and b)], they reduce the system length of the bracing member on the minimum axis to L_{d1}. The slenderness ratio, λ, should be taken as:

$\lambda = L_{d1}/r_{vv}$ for angles;

$\lambda = L_{d1}/r_{xx}$ for tubes or solid rounds (normally not critical).

Buckling should also be checked over the system length L_{d2} on the rectangular axis for buckling transverse to the bracing and, when the load is not equally split into tension and compression, over the system length L_d for the algebraic sum of the loads (see **5.3.3.1**).

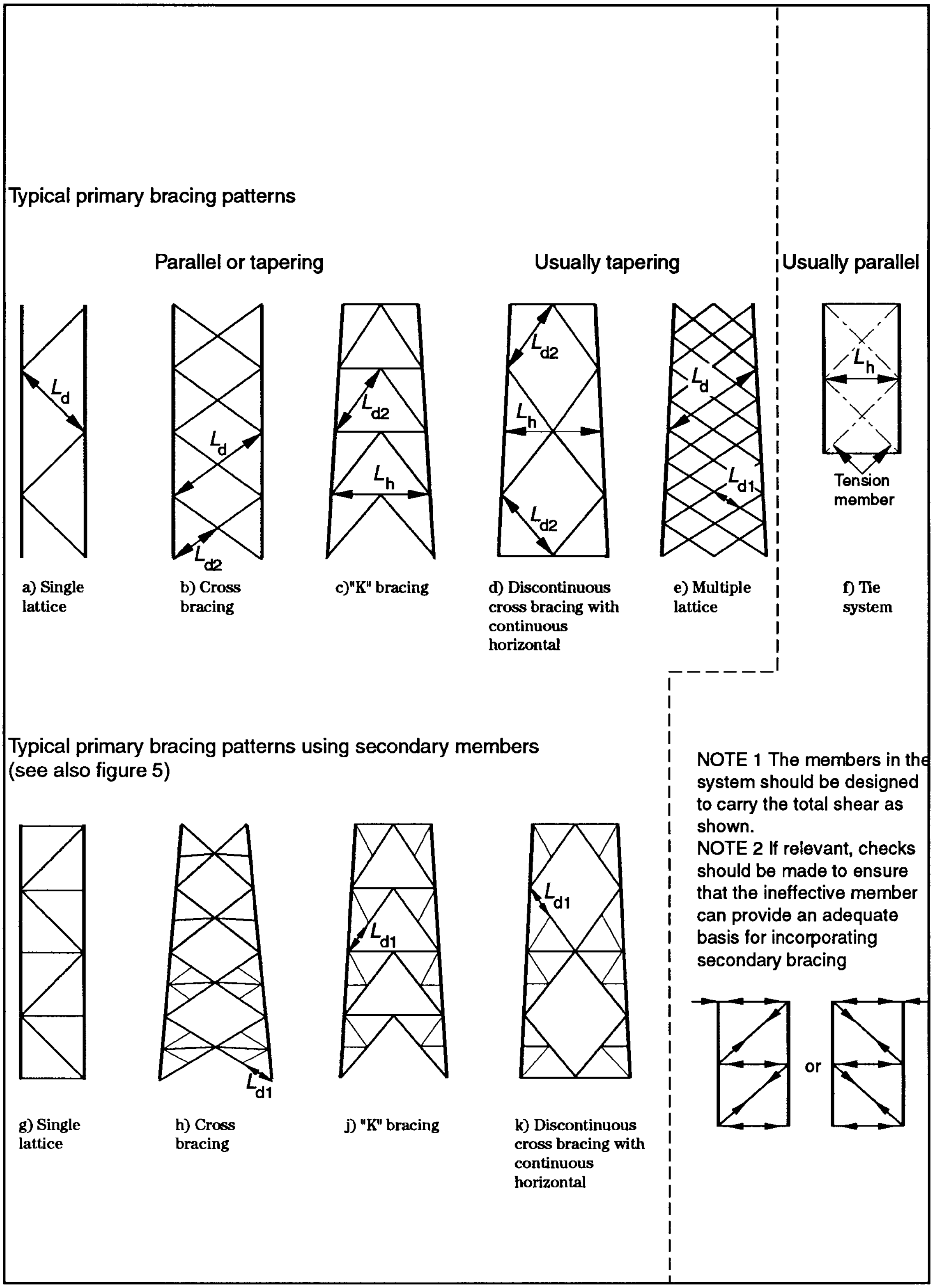


Figure 4 — Typical bracing patterns

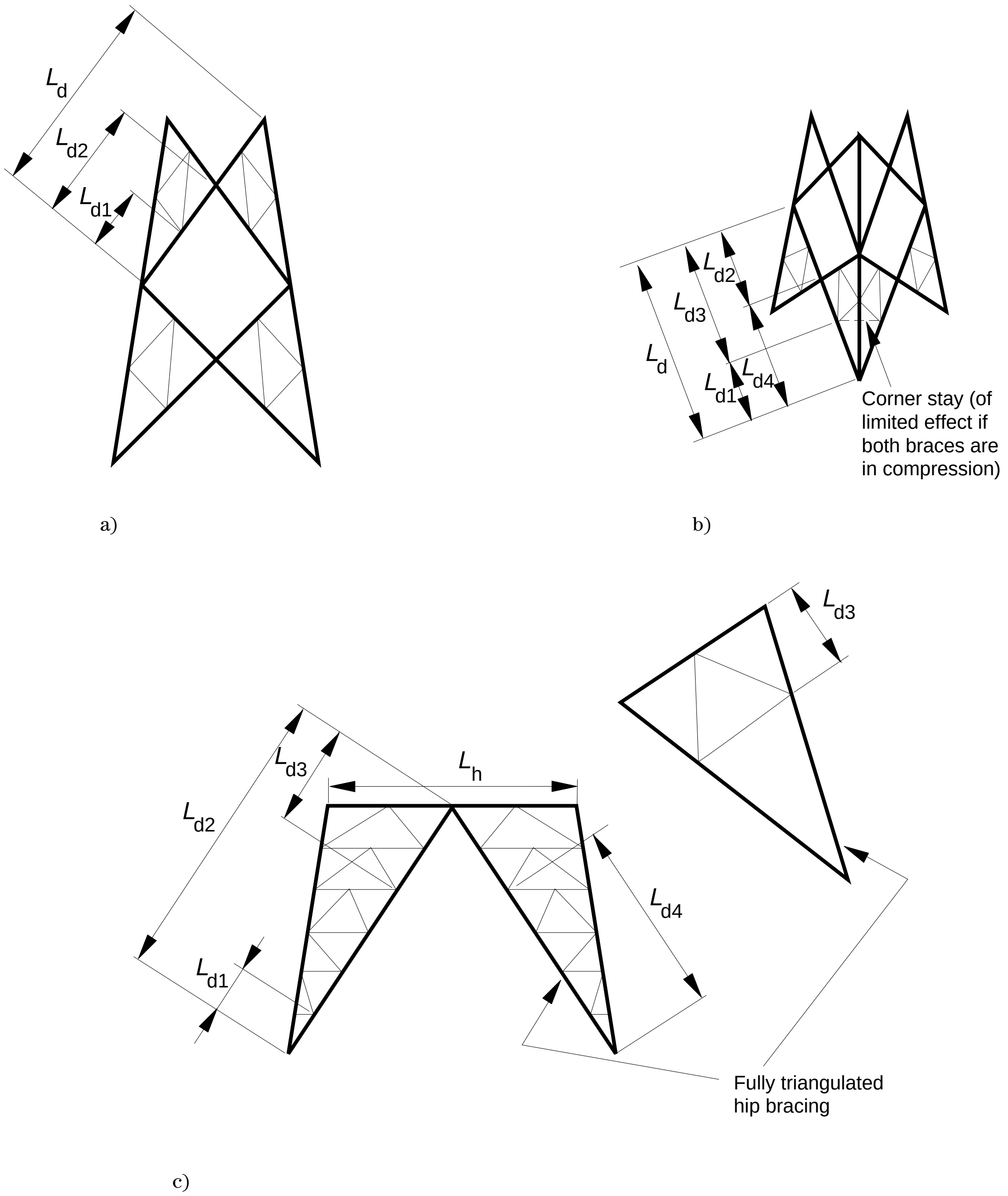


Figure 5 — Use of secondary bracing systems
[See also patterns g) to k) in Figure 4]

5.3.3.3 *Discontinuous cross bracing with continuous horizontal at centre intersection*

The diagonal members should be designed in accordance with **5.3.3.2**. 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 both members are in compression [see Figure 4d) and k)]. This criterion will be satisfied by ensuring that the horizontal member without plan bracing withstands (as a strut over its full length L_h on the rectangular axis) the algebraic sum of the load in the two members of the cross brace resolved in the horizontal direction.

NOTE A maximum effective slenderness ratio for the horizontal of 180 is good practice (as a strut over its full length on the rectangular axis).

5.3.3.4 *Cross bracing with diagonal corner stays*

In some patterns of cross bracing a corner stay is inserted to reduce the system length transverse to the plane of bracing [see Figure 5b)]. A similar procedure to that used for **5.3.3.1** may be used to determine whether the members are adequate with this restraint.

In this case five buckling checks should be carried out, related to the appropriate system length:

a) buckling of member against the maximum load over L_{d1} on the minimum axis;

b) buckling of member against the maximum load over L_{d2} on the transverse rectangular axis;

c) buckling of two members in cross brace against the algebraic sum of loads in cross brace over L_{d3} on the transverse axis;

d) 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 corner brace over L_{d4} on the transverse axis;

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

5.3.4 ***K bracing systems*** [see Figure 4c), j) and 5c)]

5.3.4.1 *Diagonal members*

The critical system length without secondary members is L_{d2} on the minimum axis and the slenderness ratio, λ, should be taken as:

$\lambda = L_{d2}/r_{vv}$ for angles;

$\lambda = L_{d2}/r_{xx}$ for tubes and solid rounds.

If secondary members are used then the system length is L_{d1} on the minimum axis, and the slenderness ratio, λ, should be taken as:

$\lambda = L_{d1}/r_{vv}$

Buckling over system length L_{d2} of the face bracing on the rectangular axis should also be checked if secondary bracing, but no hip bracing, has been provided. Thus the slenderness ratio, λ, should be taken as:

$\lambda = L_{d2}/r_{xx}$ or L_{d2}/r_{yy} as appropriate, for all sections.

Where triangulated hip bracing has been provided, then the appropriate 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 ratio, λ, should be taken as:

$\lambda = L_{d4}/r_{xx}$ or L_{d4}/r_{yy} for all sections.

5.3.4.2 *Horizontal members with plan bracing*

Where the length of the horizontal edge members becomes large, it is normal to subdivide them as part of the plan bracing which provides a convenient basis to do this. (See **5.3.5**.)

The system length of the horizontal members is taken between intersection points in the plan bracing for buckling transverse to the face of the structure, and between supports in plane for buckling in the plane of the frame.

Care is needed 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 in one plane 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.

5.3.4.3 *Horizontal members without plan bracing*

For small widths of towers and for masts, plan bracing may sometimes be omitted. [See also **5.5.1**d) for reduction factor K_1.]

The rectangular radius of gyration should be used for buckling transverse to the frame over system length L_h (see Figure 7). In addition for single angle members, the radius of gyration about the vv axis should be used over L_{h2} [see Figure 7a)] unless restraint by secondary bracing at intervals along the length is provided in which case the system length is L_{h1} [see Figure 7b)]. However, buckling about the rectangular axis will be critical except in the case of an unequal angle.

Additional allowance should be made for the bending stresses induced in the edge members by loads transverse to the frame, e.g. wind which can produce significant point loads.

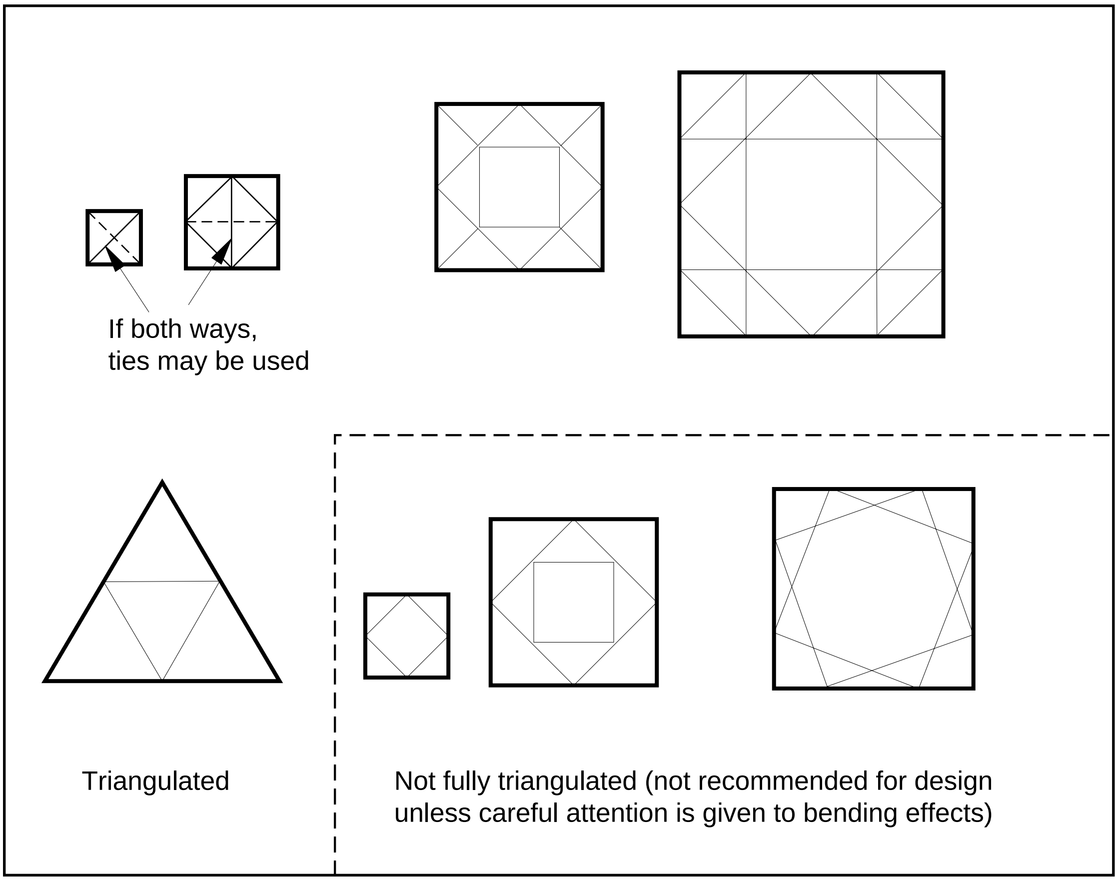


Figure 6 — Typical plan bracing

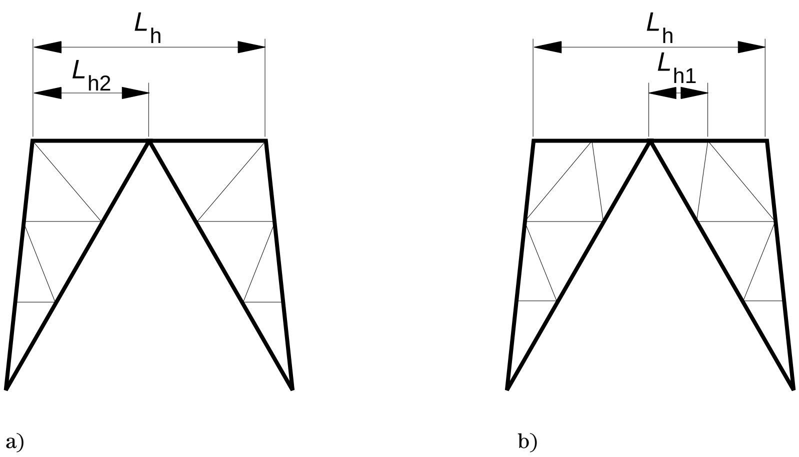


Figure 7 — K bracing horizontals without plan bracing

5.3.4.4 *Cranked K bracing*

For large tower widths, a crank or bend may be introduced into the main diagonals (see Figure 8). This has the effect of reducing the length and size of the redundant members but produces high stresses in the members meeting at the bend and necessitates fully triangulated transverse support at the joint. Diagonals and horizontals should be designed as for K bracing, with the system lengths for diagonals based on their lengths to the knee joint.

5.3.4.5 *Portal frame*

A horizontal member is sometimes introduced at the bend to turn the panel into a portal frame (see Figure 9). The main disadvantage of this is the lack of articulation present in the K brace. This system is sensitive to foundation settlement or movement and special consideration should be given to this possibility.

This example also shows special secondary bracing which is less sensitive to loads resulting from such movement.

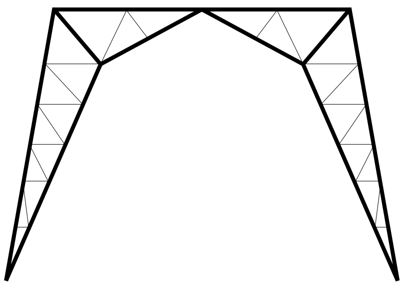


Figure 8 — Cranked K bracing

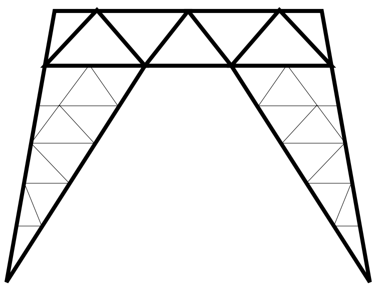


Figure 9 — Portal frame

5.3.5 *Plan bracing*

Plan bracings are required at bend lines of the leg, to provide lateral restraint to long horizontal members and to distribute local loads from ancillaries, etc.

Plan bracing is also required to maintain the shape of square towers, particularly where K bracing is used, and to distribute eccentric loads. The non-triangulated systems shown in Figure 6 do not satisfy the first requirement.

Where the plan is not fully triangulated, additional allowance should be made for the bending stresses induced in the edge members by loads, e.g. wind, transverse to the frame whereby the main diagonal can impose a point load arising from the summation of the distributed loading on the various face members.

Attention should be given to vertical bending due to the self weight of plan bracing. Support from hip bracing can be used to reduce long spans. The design should be detailed to eliminate any face slope effect which promotes downward displacement of inner plan brace members.

5.3.6 *Multiple lattice bracing*

Multiple lattice bracing members should be designed under the applied load on the minimum system L_{d1} for the slenderness ratio $\lambda = L_{d1}/r_{vv}$. The angle bracing members of a multiple lattice configuration should be checked on a system length from leg to leg with the appropriate radius of gyration r_{xx} or r_{yy} [see Figure 4e)] such that the overall effective slenderness ratio is limited to 350.

For the stability of the panel r_{xx}/r_{vv} should be greater than 1.50, where r_{xx} is the radius of gyration about the axis parallel to the plane of the lattice.

5.3.7 *Tie systems*

Great care is needed with the use of tie systems particularly if used on large masts or towers, on masts or towers with sloping legs or in conjunction with secondary members. Each diagonal member of a pair of cross bracing members [see Figure 4f)] should be capable of carrying the full bracing shear load in tension. Detailing to give an initial tension within the bracing and to provide mutual support at the central cross will be required to minimize deflection. It will also be necessary to pay special attention to possible fatigue problems.

The horizontal members should be designed as horizontal strut members with or without plan bracing as appropriate for compression along their whole length.

NOTE 1 Tension systems are very sensitive to methods of erection and to modification or relative movements. Special attention should be paid to possible excessive deflections and fatigue. To ensure their effectiveness measures should be taken to ensure that these matters are carefully considered.

NOTE 2 A maximum effective slenderness ratio of 350 for tension members is good practice.

5.3.8 *Compound members*

5.3.8.1 *Double angles*

Compound members consisting of a pair of identical angles back to back (forming a T), separated by a small distance and connected at intervals by spacers and stitch bolts, may be used for bracing. They should be checked for buckling about both rectangular axes as follows. (See also **5.5.1**.)

a) *For buckling about the xx axis* (see Figure 2) the two angles should be assumed to act compositely for the purpose of calculating stiffness and radius of gyration.

b) *For buckling about the yy axis* (see Figure 2) the additional deformation due to shear should be taken into account by modifying the slenderness ratio, λ, in accordance with the following:

$$\lambda^2 = \lambda_0{}^2 + \lambda_1{}^2 \text{ taking a maximum value of } \lambda_1 = 0.75\lambda_0$$

where

λ_0 is the slenderness ratio of the full compound member;

λ_1 is the slenderness ratio of one component angle ($\lambda_1 = C/r_{vv}$).

In order to keep the effect of this interaction to a minimum, the spacing between stitch bolts should be limited to give a maximum value λ_1 of 90.

When only stitch bolts and packs are used, composite section properties should be based on either:

1) the actual space between the individual angle members, or

2) a space taken as 1.5 times the thickness of one of the angle members,

whichever is the smaller. If batten plates are used, composite section properties may always be used based on the actual space between the individual angle members.

5.3.8.2 *Other compound members*

For other compound member configurations and the sizing of battens and stitch bolts or welds, refer to **5.2.2**.

5.4 Secondary members

Typical secondary bracing arrangements are shown in Figures 4 to 9.

NOTE A maximum effective slenderness ratio of 240 for secondary bracing members is good practice. The use of high slenderness can lead to the possibility of individual members vibrating, and can make them vulnerable to damage due to bending from local loads.

In order to design secondary members it is necessary to apply a hypothetical force acting transverse to the leg member (or other chord if not a leg) being stabilized at the node point of the attachment of the secondary member. This force varies with the slenderness of the leg member being stabilized and is expressed as a percentage of the leg load, F for which the following two checks should be carried out.

a) Values of this percentage of the leg load for various values of the slenderness of the leg should be taken from Table 1. The slenderness ratio to be used should be that of the restrained member, which will normally be L/r_{vv}, where L is the length between nodes and r_{vv} is the minimum radius of gyration.

The effect of applying this force in the plane of the bracing at each node in turn should be calculated.

b) When there is more than one intermediate node in a panel then the secondary bracing systems should be checked separately for 2.5 % of the leg load shared equally between all the intermediate node points. These loads should be assumed to act together and in the same direction, i.e. at right angles to the leg and in the plane of the bracing system.

In both cases a) and b) the distribution of forces within the triangulated secondary bracing panel should then be determined by stress diagrams or linear elastic analysis.

Table 1 — Applied force as percentage of leg load, *F*

Slenderness ratio, λ	Applied force (percentage of leg load, F) %
0 to 40	1.02
45	1.15
50	1.28
55	1.42
60	1.52
65	1.60
70	1.65
75	1.70
80	1.75
85	1.80
90	1.85
95	1.92
100	2.00
105	2.08
110	2.16
115	2.23
120	2.30
125	2.38
130	2.46
>132	2.50

NOTE 1 The tabulated values assume minimum axis restraint of the leg by two such forces each in the plane of the bracing. Where the critical direction is in the plane of the bracing (e.g. round or cruciform sections) the values should be multiplied by a factor of $\sqrt{2}$.

NOTE 2 These loads are not additive to the existing forces on the tower or mast. 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 this figure may be unsafe and a more refined value should be obtained by taking into account the eccentricity moment and secondary stresses arising from leg deformation.

NOTE 3 In general if the bracing strength requirements are met, the stiffness requirements should be sufficient.

5.5 Calculation of effective slenderness Λ_{eff}

5.5.1 *General*

In order to calculate the design buckling resistance of the member, the effective slenderness Λ_{eff} should be determined.

The effective slenderness Λ_{eff} should be derived from:

$$\Lambda_{eff} = K\Lambda$$

where

Λ is the relative slenderness of the member about the appropriate axis for which the strength is required and is given by:

$$\Lambda = \lambda/\lambda_1$$

where

λ is the slenderness ratio obtained from **5.3**;

$$\lambda_1 = \pi\left[\frac{E}{\sigma_r}\right]^{0.5}$$

$$= 85.8\left[\frac{275}{\sigma_r}\right]^{0.5} \text{ when } E = 205\,000 \text{ Mpa}$$

σ_r is a reference stress given in **6.3**;

K is an effective slenderness factor depending on the structural configurations and is given as follows.

a) *Leg members*

K is given in Table 2.

b) *All bracing members except where c) and d) below are applicable*

K is dependent on both the bracing pattern (see Figures 4 and 5) and the connections of the bracing to the legs. K is dependent on the relative stiffness of the adjoining members, of the bolt or weld group and of the connecting gusset plate if relevant. Values of K for typical configurations are given in Table 3. For welded connections in particular, the factors depend on continuity of the weld all round the joined member(s).

c) *Compound bracing members buckling about the yy axis of the compound member*

K is taken as 1.0.

d) *Horizontal bracing members in K bracing without plan bracing*

Horizontal members of K bracing without plan bracing (see **5.3.4.3**) usually have compression in one half of their length and tension in the other. In such cases the effective slenderness factor K of the horizontal normal to the plane of the frame, determined from Table 3 ignoring continuity, should be multiplied by the factor K_1 given in Table 4, depending on the ratio of tension load P_2 to the compression load P_1.

Table 2 — Effective slenderness factor K for leg members

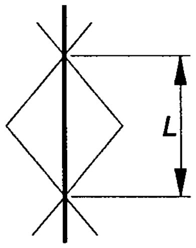
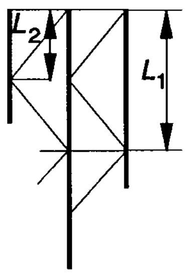
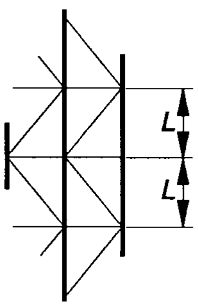
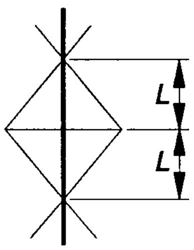
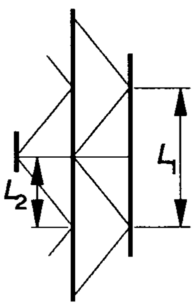
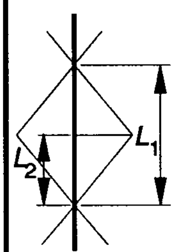
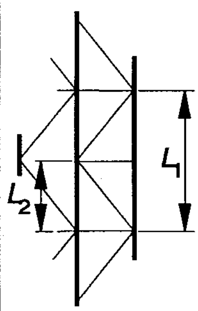
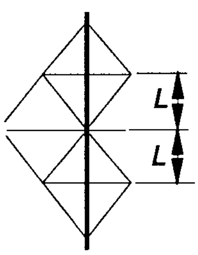
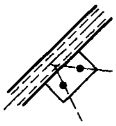
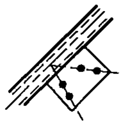
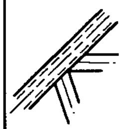
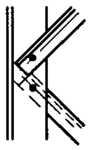
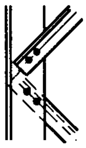
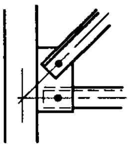
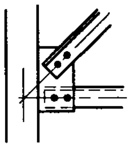
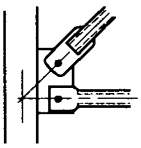
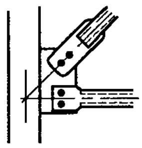
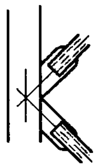
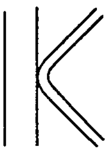
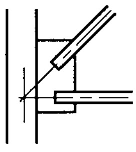
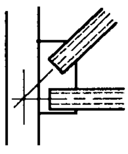
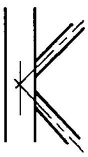
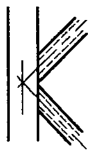
<table>
<tr><th colspan="3">Symmetrical bracing</th><th colspan="4">Asymmetrical bracing</th></tr>
<tr><th></th><th>∟</th><th>+ ● ○</th><th></th><th>∟</th><th></th><th>+ ● ○</th></tr>
<tr><th>Axis</th><th>v - v</th><th>y - y</th><th>Axis</th><th>v - v</th><th>y - y</th><th>y - y</th></tr>
<tr>
<td>
Primary bracing at both ends</td>
<td>$0.8 + \frac{\Lambda}{10}$
but ≥ 0.9 and ≤ 1.0</td>
<td>1.0
NOTE. If primary bracing at both ends is double bolted 0.9 may be used, or if welded 0.85</td>
<td>
Discontinuous top end with horizontals</td>
<td rowspan="2">For L_2[a]
$1.2\left(0.8+\frac{\Lambda}{10}\right)$
but ≥ 1.08 and ≤ 1.2</td>
<td rowspan="2">For L_1
$1.2\left(0.8+\frac{\Lambda}{10}\right)$
but ≥ 1.08 and ≤ 1.2</td>
<td rowspan="2">For L_1
1.0
NOTE. If primary bracing at both ends is double bolted 0.9 may be used, or if welded 0.85</td>
</tr>
<tr>
<td rowspan="2">
Asymmetric

Symmetric
Primary bracing at one end and secondary bracing at the other</td>
<td rowspan="2">$0.8 + \frac{\Lambda}{10}$
but ≥ 0.9 and ≤ 1.0</td>
<td rowspan="2">1.0
NOTE. If primary bracing at both ends is double - bolted 0.95 may be used, or if welded 0.9</td>
<td>
Primary bracing at both ends</td>
</tr>
<tr>
<td rowspan="2">

Primary bracing at both ends of L_1</td>
<td rowspan="2">For L_2[a]
$0.8 + \frac{\Lambda}{10}$
but ≥ 0.9 and ≤ 1.0</td>
<td rowspan="2">For L_1
$0.8 + \frac{\Lambda}{10}$</td>
<td rowspan="2">For L_1
1.0
NOTE. If primary bracing at both ends is double bolted 0.9 may be used, or if welded 0.85</td>
</tr>
<tr>
<td>
Secondary bracing at both ends</td>
<td>$0.8 + \frac{\Lambda}{10}$
but ≥ 0.9 and ≤ 1.0</td>
<td>1.0</td>
</tr>
<tr><td colspan="7">[a] Only critical if a very unequal section is used.</td></tr>
</table>

Table 3 — Effective slenderness factor K for bracing members

Legs	Single bolted angles and tubes Continuous welded rod		Double bolted angles and tubes Individual welded rod		Welded tubes	
	Type	K[a]	Type	K[a]	Type	K[a]
Any	 Continuous	$0.7 + 0.35/\Lambda$ all directions				
	 Continuous	1.0	 Continuous	0.9	 Continuous	0.85
Angle legs		*Normal to the frame*[c] Discontinuous both ends $0.7 + 0.58/\Lambda$ Continuous one end $0.7 + 0.40/\Lambda$ *All other directions* $0.7 + 0.35/\Lambda$		As for single bolted angles		
Tube or rod legs						
		0.95[b]		In - plane of the frame 0.85 - all other directions 0.95[b]		In - plane of the frame 0.85 - all other directions 0.95[b]
		0.9		In - plane of the frame 0.80 - all other directions 0.85		In - plane of the frame 0.80 - all other directions 0.85
				0.8		0.8

[a] Where ends are not the same an average "K" value should be used.
[b] This factor assumes a geometric reduction to the system length only.
[c] See also **6.1** which limits the buckling resistance of single bolted angles.

Table 4 — Factor K_1 for horizontal of K brace (without plan bracing)

Ratio P_2/P_1	Factor K_1
0.00	0.73
0.20	0.67
0.40	0.62
0.60	0.57
0.80	0.53
1.00	0.50

6 Calculation of design strength for members in compression

6.1 General

The design buckling resistance, N, of a compression member should be taken as:

$$N = j\chi A\sigma_r/\gamma_m$$

where

- j is 0.8 for single angle members connected by one bolt at each end, 0.9 for single angle members connected by one bolt at one end and continuous at the other end and 1.0 in all other cases;
- A is the cross-sectional area of the member;
- χ is the reduction factor for the relevant buckling mode, given in **6.2**;
- σ_r is a reference stress given in **6.3**;
- γ_m is the partial factor on strength, as given in BS 8100-1 and -4, appropriate to the quality class of the structure, subject to the following.

  a) For untested structures:

  - γ_m is 1.1 for Class A structures;
  - γ_m is 1.2 for Class B structures;
  - γ_m varies from about 1.3 to 1.45 for Class C structures depending on the performance requirements.

  b) For angle section towers which have successfully been subjected to fullscale tests under the equivalent factored loading or where similar configurations have been type tested:

  - γ_m is 1.0 for Class A structures;
  - γ_m is 1.1 for Class B structures;
  - γ_m varies from about 1.2 to 1.35 for Class C structures depending on the performance requirements.

For hot rolled steel members with the types of cross-section commonly used for compression members, the relevant buckling mode is general “flexural” buckling.

In some cases the “torsional” or “flexural-torsional” modes may govern. For information on the design buckling resistance of compression members for those sections not covered in this part of BS 8100, reference may be made to BS 5950-5.

6.2 Reduction factor χ

For constant axial compression in members of constant cross-section, the value of χ for the appropriate effective slenderness Λ_{eff}, should be determined from:

$$\chi = \frac{1}{\varphi + [\varphi^2 - \Lambda_{eff}^2]^{0.5}}$$

where

- φ is $0.5\,[1 + \alpha\,(\Lambda_{eff} - 0.2) + \Lambda_{eff}^2]$;
- α is an imperfection factor;
- Λ_{eff} is obtained from **5.5.1**.

and where

$$\chi \leq 1$$

The imperfection factor α corresponding to the appropriate buckling curve should be obtained from Table 5.

Table 5 — Imperfection factors

Buckling curve	Imperfection factor α
a	0.21
b	0.34
c	0.49
d	0.76

Values of the reduction factor χ for the appropriate effective slenderness Λ_{eff} may be obtained from Table 6.

6.3 Calculation of reference stress

The reference stress σ_r required to derive the ultimate stress of a member depends on the slenderness of the section. Most hot rolled sections are at least semi-compact. Values of μ and the limiting B/t and D/t ratios are given below, together with formulae for the reference stress σ_r which is used in place of the yield stress σ_y for the design of slender sections.

a) *Hot rolled angle sections*

$\sigma_r = \sigma_y$ if $B/t \leq \mu$

$\sigma_r = \sigma_y(2 - B/\mu t)$ if $\mu < B/t < 1.33\mu$

$\sigma_r = \pi^2 E/(5.1B/t)^2$ if $B/t \geq 1.33\mu$

Table 6 — Reduction factors χ

Λ_{eff}	Buckling curve			
	a	b	c	d
0.2	1.0000	1.0000	1.0000	1.0000
0.3	0.9775	0.9641	0.9491	0.9235
0.4	0.9528	0.9261	0.8973	0.8504
0.5	0.9243	0.8842	0.8430	0.7793
0.6	0.8900	0.8371	0.7854	0.7100
0.7	0.8477	0.7837	0.7247	0.6431
0.8	0.7957	0.7245	0.6622	0.5797
0.9	0.7339	0.6612	0.5998	0.5208
1.0	0.6656	0.5970	0.5399	0.4671
1.1	0.5960	0.5352	0.4842	0.4189
1.2	0.5300	0.4781	0.4338	0.3762
1.3	0.4703	0.4269	0.3888	0.3385
1.4	0.4179	0.3817	0.3492	0.3055
1.5	0.3724	0.3422	0.3145	0.2766
1.6	0.3332	0.3079	0.2842	0.2512
1.7	0.2994	0.2781	0.2577	0.2289
1.8	0.2702	0.2521	0.2345	0.2093
1.9	0.2449	0.2294	0,2141	0.1920
2.0	0.2229	0.2095	0,1962	0.1766
2.1	0.2036	0,1920	0,1803	0,1630
2.2	0.1867	0.1765	0.1662	0.1508
2.3	0.1717	0.1628	0.1537	0.1399
2.4	0.1585	0.1506	0.1425	0.1302
2.5	0.1467	0.1397	0.1325	0.1214
2.6	0.1362	0.1299	0.1234	0.1134
2.7	0.1267	0.1211	0.1153	0.1062
2.8	0.1182	0.1132	0.1079	0.0997
2.9	0.1105	0.1060	0.1012	0.0937
3.0	0.1036	0.0994	0.0951	0.0882

where

σ_y is the specified minimum yield stress of material of member;

B is the leg length of the angle. For unequal angles and compound angles, B is the longer length, except for single angles connected by bolts or welding through one leg only at both ends of the element under consideration for which B should be taken as the length of the connected leg;

t is the thickness of angle leg;

μ is $0.567\sqrt{\left(\dfrac{E}{\sigma_y}\right)}$;

E is the modulus of elasticity.

b) *Hot rolled tubular sections*

$\sigma_r = \sigma_y$ if $D/t \le \mu$

$\sigma_r = 0.10\left(\dfrac{E}{D/t}\right)$ if $D/t > \mu$

where

σ_y is the specified minimum yield stress of material of member;

D is the diameter of tubular section;

t is the wall thickness of tubular section;

μ is $0.10\left(\dfrac{E}{\sigma_y}\right)$;

E is the modulus of elasticity.

c) *Cold formed angle sections*

$$\sigma_r = \sigma_y \quad \text{if } B/T \leq \mu$$

$$\sigma_r = \sigma_y \left(\frac{5}{3} - \frac{2}{3} B/\mu t\right) \quad \text{if } \mu < B/t < 1.5\mu$$

$$\sigma_r = \pi^2 E/(5.1\ B/t)^2 \quad \text{if } B/t \geq 1.5\mu$$

where

σ_y is the specified minimum yield stress of material of member after forming;

B is the leg length of the angle as defined in **6.3**a).

t is the thickness of angle leg;

μ is $0.503\sqrt{\left(\frac{E}{\sigma_y}\right)}$;

E is the modulus of elasticity.

d) *Cold formed tubular sections*

$$\sigma_r = \sigma_y \quad \text{if } D/t \leq \mu$$

$$\sigma_r = 0.10\left(\frac{E}{D/t}\right) \quad \text{if } D/t > \mu$$

where

σ_y is the specified minimum yield stress of material of member;

D is the diameter of tubular section;

t is the wall thickness of tubular section;

μ is $0.10\left(\frac{E}{\sigma_y}\right)$;

E is the modulus of elasticity.

6.4 Selection of buckling curves

For flexural buckling the appropriate buckling curves should be determined from Table 7.

7 Calculation of resistance of members in tension

The resistance of a member in tension is the product of the yield strength and the net sectional area.

The net sectional area of an angle connected through one leg should be taken as the net area of the connected leg, i.e. gross area minus holes, plus half the area of the unconnected leg.

The net sectional area of an angle suitably connected through both legs should be taken as the sum of the net areas of both legs, i.e. gross area minus holes.

8 Bolted connections

8.1 Angle bolted connections

Bolted connections should be designed in accordance with BS 5950-1 but with the modifications that the design resistance of a bolt Q_u, should be the least of the following (see Figure 10).

In **8.1**a) to **8.1**f) the partial factor on strength γ_m for both tested and untested structures should be taken as given in **6.1**a) for untested structures (i.e. no advantage can be taken for structures which have been tested).

a) *Design shear resistance*

For ultimate loads this is related to the minimum tensile strength σ_t or the yield strength of the bolt steel σ_b and the relevant cross-sectional area A_s:

$Q_u = 0.65\sigma_t A_s/\gamma_m$ or $0.95\sigma_b A_s/\gamma_m$ whichever is the lesser.

b) *Design bearing resistance*

The design bearing resistance is normally related to the acceptable amount of deformation in the hole and whilst values up to three times yield can be used without failure, it is recommended that the stress is limited to twice the yield strength of the member steel, σ_y, to reduce the deformation to acceptable limits, i.e:

$Q_u = 2.0Dt\sigma_y/\gamma_m$

c) *Design tensile resistance*

When prying is ignored:

$Q_u = 0.56\sigma_t A_s/\gamma_m$ but not greater than $0.8\sigma_b A_s/\gamma_m$

When prying is taken into account in accordance with Table 8:

$Q_u = 0.7\sigma_t A_s/\gamma_m$ but not greater than $\sigma_b A_s/\gamma_m$

d) *End distance of bolt x*

$$Q_u = 1.33\ \sigma_y\ xt/\gamma_m \quad \text{for } x > 1.5D$$

$$Q_u = 2.0\ \sigma_y\ (x - D/2)\ t/\gamma_m \quad \text{for } x \leq 1.5D$$

e) *Centres of bolts for multiple bolt connections*

$Q_u = \sigma_y\ (z - D/2)\ t/\gamma_m$

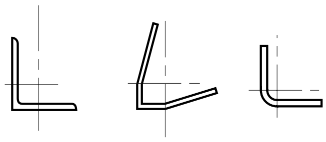
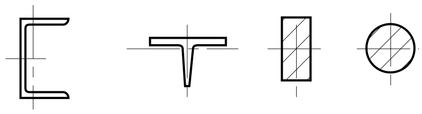
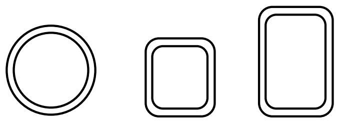
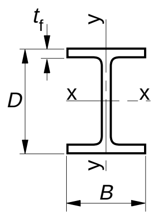
f) *Edge distance of bolt*

$Q_u = 2.67\ \sigma_y\ (y - D/2)\ t/\gamma_m$

NOTE 1 To utilize a design bearing resistance of $2.0\,Dt\sigma_y$ requires $x \geq 1.5D$ and $y \geq 1.25D$ and $z \geq 2.5D$. This is after allowance has been made for fabrication tolerances in sizes.

NOTE 2 The design bearing resistance given in b) above may also be used when the thread is in bearing.

Table 7 — Selection of buckling curve for a cross-section

	Cross section	Limits	Buckling about axis	Buckling curve
Commonly used sections	Angle sections 	hot rolled	any	b
		cold rolled - using f_{yb} (See NOTE 1)	any	c
		cold formed - using f_{ya} (See NOTE 1)	any	c
	C, T, and solid sections 		any	c
	Hollow sections 	hot rolled	any	a
		cold rolled - using f_{yb} (See NOTE 1)	any	b
		cold formed - using f_{ya} (See NOTE 1)	any	c
Other sections	Rolled 'I' sections 	D/B ≤ 1.2: $t_f \leq 40$ mm	x - x y - y	a b
		40 mm < $t_f \leq 100$ mm	x - x y - y	b c
		D/B ≤ 1.2: $t_f \leq 100$ mm	x - x y - y	b c
		$t_f > 100$ mm	x - x y - y	d d

NOTE 1 f_{yb} is the basic yield of the material before cold forming ;
f_{ya} is the average yield stress of the material after cold forming as calculated using BS 5950-5

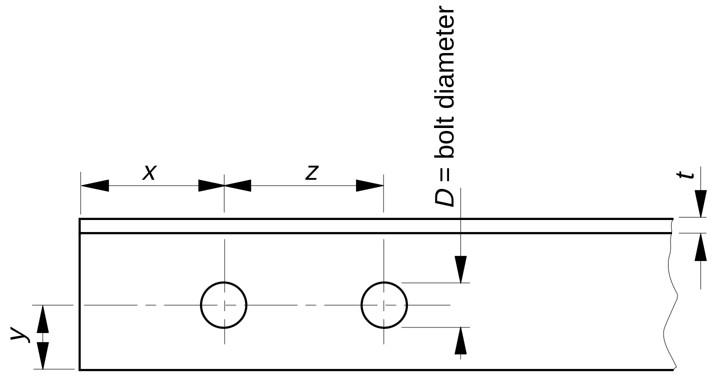


Figure 10 — Location of bolts

Table 8 — Relationships between flange, tube and bolts

Ratio of flange to tube wall thickness b for						
Blank flanges	2.0	1.9	1.8	1.7	1.6	1.5
Ring flanges	3.0	2.8	2.6	2.4	2.2	2.0
Ratio of tensile design strength required to maximum tensile capacity of tube[a]	Increase in direct tension to allow for prying in the flange bolts (bolts prying factor)					
1.0	1.2					
0.9	1.1	1.2				
0.8	1.0	1.1	1.2			
0.7	1.0	1.0	1.1	1.2		
0.6	1.0	1.0	1.0	1.1	1.2	
0.5	1.0	1.0	1.0	1.0	1.1	1.2

Do not use in shaded zone

[a] The relationship between flange thickness ratio and bolt prying factor is a function of the extent to which the tube is loaded in tension. In permissible stress terms this is f_t / p_t or $F\gamma_m / A\sigma_y$ in accordance with this part of BS 8100.

[b] Where different grades of steel are used in the tube and flange, the minimum ratio of flange to tube wall thickness should be varied in accordance with the ratio of the square root of their respective yields, within the range shown in the table.

8.2 Tubular and solid round bolted connections

Where connections are formed with gusset plates and bolts in shear, the rules in **8.1** should be used. Details which are not covered in **8.1** should be designed in accordance with BS 5950-1.

Where round flange joints are used on tubular members, the following design recommendations should be followed.

a) The relationships between flange thickness expressed as a ratio of tube wall thickness, bolt prying factor, and extent to which the tube is loaded in tension are shown in Table 8.

For the required tube design tensile strength expressed as a proportion of its maximum tensile capacity, the combination of bolt prying factor along the line in the table, and flange thickness ratio above this value should be provided as a minimum.

b) The tensile design strength of the bolts should be divided by the bolt prying factor shown in Table 8 to determine the direct tension capacity of the bolt.

c) The pitch circle diameter of the bolts should be as small as practicable.

d) The “edge” distance between the flange edge and pitch circle diameter should not normally be less than 1.25 times the bolt diameter.

NOTE It may be necessary to increase the minimum thickness to compensate for welding distortion, particularly in the case of blank flanges, where a seal or snug fit is required.

9 Mast guy assemblies

9.1 General

9.1.1 *Requirements of a mast guy assembly*

The basic requirement of a mast guy assembly is that it should provide a horizontal support of the greatest possible stiffness, whilst increasing the superimposed load on the structure as a whole by the minimum amount. The guy assembly should also be an economical solution to the support problems when considered in the light of initial cost, erection and maintenance throughout its life.

The guy assembly will normally consist of the following items: the guy rope, terminations and a linkage system at the lower end to permit coarse and fine adjustment of the length (tension) and to facilitate the replacement of the guys.

9.1.2 *Guys*

Guys should generally be made up of steel wires with as large a diameter as practical bearing in mind the other characteristics required.

The detailed make-up and form of construction of guys and their constituent parts should conform to the requirements of the appropriate British Standard or Euronorm.

BS 302-1 to -8 covers a range of galvanized steel wire ropes for engineering use. Those with steel cores and adequate wire diameters may be suitable for mast guys. Spiral strands and locked coil are not covered and should be obtained from an experienced manufacturer in accordance with this part of BS 8100.

The make-up and form of construction should take into account the following:

a) the required strength of the guy and its attachments;

b) the sensitivity of the guy to dynamic excitation by wind or otherwise;

c) the required stiffness of the guy (both axial and flexural);

d) the requirement for corrosion resistance of the guy;

e) the requirement for maintenance and replacement of the guy;

f) the requirement of dielectric properties;

g) the requirement for fatigue resistance.

9.2 Materials for mast guys

9.2.1 *Wire for strands*

The wire for strands should be hard drawn steel wire conforming to BS 2763 and hot-dip galvanized to withstand the tests in BS 443. Stainless steel should conform to BS 1554.

Where wire sizes are outside the scope of the British Standard, tests should be agreed with the maker and the engineer before manufacture is commenced.

9.2.2 *Materials for sockets*

Sockets should be cast or forged in accordance with the BS 463.

Where cast steel sockets are used, freedom from defects in the casting should be ensured by an acceptable non-destructive test or manufacturers’ certificate.

The filling material for the sockets should be such that creeping of the loaded guy through the socket is prevented. Socketing should be carried out in accordance with BS 7035.

The filling material should be chosen from one of the following:

a) white metal conforming to BS 643;

b) zinc;

c) epoxy resin mix specifically designed for this purpose.

9.2.3 *Non-metallic guys*

Synthetic materials are generally proprietary products and reference should be made to manufacturers’ data for properties. Careful selection is necessary with these materials as the low modulus of elasticity of some products makes it dangerous to substitute them on a strength for strength basis with steel guys.

They can also require higher initial tensions to compensate for low stiffness which can lead to high frequency vibrations.

9.3 Mechanical properties

9.3.1 *Strength of wires, ropes and strands*

The strength of steel wire ropes and strands can be obtained from manufacturers’ catalogues up to a diameter of about 70 mm.

The characteristic strength of wire may be taken as the specified nominal values of the breaking stress.

The characteristic value of the 0.2 % proof stress of wire may be taken as the specified nominal value.

The characteristic strength of a parallel wire strand or locked coil strand may be taken as the aggregate characteristic strength of the constituent wires of the strand.

The characteristic strength of a spiral strand of lay length not less than ten times the strand diameter may be taken as 95 % of the aggregate characteristic strength of the constituent wires of the strand.

The characteristic strength of a wire rope may be taken as 90 % of the characteristic aggregate strength of the constituent wires of the rope.

9.3.2 ***Modulus of elasticity***

The effective elastic modulus of guys should be used in structural analysis.

The effective elastic modulus should be determined from manufacturers' data or by testing strands or steel wire ropes of the actual make-up used in the design. Typical values for various guy constructions for preliminary purposes only are shown in Table 9, based on the actual metallic area in the cross section. If the measured value differs from the assumed value by 10 % the design calculations should be changed.

For non-metallic ropes special attention needs to be paid to their generally low modulus of elasticity.

NOTE Until a helically wound strand or steel wire rope of any type has bedded down (either by prestretching in a fabricators' shop, or by stressing during erection) the effective elastic modulus is indeterminate and generally significantly less on first loading than the values given in Table 9. Furthermore, the strain caused by first loading is not generally 100 % recoverable.

9.4 Prestretching

It is recommended that guys be prestretched prior to measuring and terminating at the supplier's works in order to ensure that the rope is in a truly elastic condition as it reduces the amount of retensioning necessary during the early life of the mast. Prestretching should be carried out by loading the guy cyclically between 10 % and 40 % of its breaking load. The number of cycles should not be less than ten. The process should not be carried out by passing the loaded guy around a sheave wheel. Parallel wire strand does not need prestretching and the process is less effective on small diameter ropes. Prestretching is most necessary when using multi-stranded ropes in excess of 20mm diameter and to a lesser extent spiral and locked coil ropes.

9.5 Terminations

The terminations used on guy ropes will depend upon the type of construction and diameter of the guys together with the service conditions applicable to the structure as a whole. Many types of terminations have been developed, some of which are covered by British Standards and are therefore of known efficiency.

Sockets should conform to BS 463 and socketing should be carried out in accordance with BS 7035.

Tests to destruction will rarely be required, but a proof load of 40 % of the breaking load of the rope should be applied to each completed termination. Terminations commonly used are shown in annex B with efficiency values shown for guidance. Values should be obtained from manufacturers.

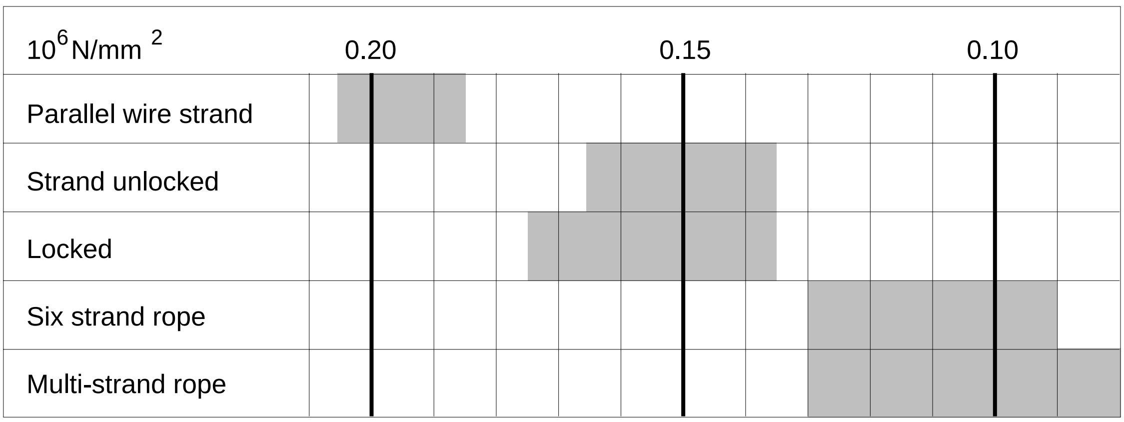
9.6 Linkages and anchorages

9.6.1 ***Guy linkage***

Each guy assembly should incorporate a linkage system to enable coarse and fine adjustments to be made to the length (tension) and to facilitate installation or changing. At the bottom of the linkage system a means of articulation in both the horizontal and vertical planes should be provided. It is desirable to provide articulation in horizontal and vertical planes at the top end of the guy assembly where it connects onto the mast column. As a general rule the total length of adjustment should be ± 0.25 % for prestretched ropes and $^{+0.25}_{-0.75}$ % for non-prestretched ropes. Due consideration should be given to lateral movements of guy in service when sizing link plates and pins. Where prising action can occur only a bolt and locked nut should be used to make the connection.

Guy link plates should be sized to provide reasonable lateral stiffness. Plates where the width to thickness ratio is about 8 should provide a satisfactory set of linkage.

Table 9 — Range of modulus of elasticity for steel wire guys



Standard lifting gear components such as thimbles conforming to BS 463, rigging screws and turnbuckles conforming to BS 4429 and steel shackles conforming to BS 6994 can be used for linkage.

NOTE 1 Unless specified in the relevant British Standard the breaking load should be obtained from the manufacturer.

The enhanced material factor specified in BS 8100-4 for guy assemblies in part provides for the wider strength variation and inspection difficulties in this area as well as the more dynamic nature of the load. It also recognises that for ropes and linkage components with guaranteed breaking loads, this actual failure load is defined as their characteristic strength.

For linkage components where a proven breaking load is not documented, their capacity may be assessed in accordance with BS 5950-1. In this case, to satisfy the first enhancement requirement above, and to ensure consistent failure levels, γ_m should be factored by γ_G, where:

$\gamma_G = 1.25$

NOTE 2 Connections in shear should be considered as pinned and be checked for bending.

NOTE 3 A series of tests to destruction for designed linkage components should achieve a breaking load at least $1.8\gamma_m$ times greater than the design tensile strength required (e.g. as for the guys).

9.6.2 *Guy anchorages*

The anchor strap should be sufficiently stiff above and below the top surface of the concrete to prevent lateral flexing and cracking of the concrete due to lateral movements induced by the guy.

9.7 Tensioning

Still air tensions should be selected such that for each guy the tension is about 10 % of its breaking load, in order to minimize the possibility of guy vibrations (see BS 8100-4:1995, **5.1.7**).

After erection, the guys should be tensioned in accordance with the design calculations.

Guys with still air tensions lower than 10 % will be more susceptible to low frequency vibrations whereas guys with higher still air tensions will be more susceptible to high frequency vibrations.

9.8 Guy dampers

9.8.1 *General*

To suppress guy vibrations that can occur under wind, as described in BS 8100-4:1995, **5.1.7**, the following procedures should be followed.

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

Where guy dampers are not fitted the guys should be carefully observed during the first year of service to ensure that excessive oscillations are not occurring.

9.8.2 *Control of response to vortex excitation*

Stockbridge type vibration dampers have a long and successful history of use to minimize response to vortex shedding, and are suitable for most installations. They should be purchased from an experienced manufacturer. Proprietary developments of this type are also available.

It is necessary to specify the frequency band of vibration. If no direct evidence is available, coverage should be considered for the range between $n_1 = 350/D$ and $n_2 = 9 \times 10^3\sigma_i/\sigma_u\, D$ where n is the frequency in Hz, D is the diameter in mm and σ_i is the initial tensile stress. This may require more than one damper.

Other types of damper may be used, including telescopic shock absorbers or rope loops clamped to the guys. Spiral plastic dampers can be effective for light guys. In such cases separate dampers for orthogonal directions may be required to ensure protection against vibration in any plane. Alternative aerodynamic protection is discussed in BS 8100-4:1995, **5.1.7**.

9.8.3 *Control of galloping excitation*

There is evidence that partial control of galloping may be obtained by the attachment of a hemp 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.

Helical spoilers formed out of preformed or clamped on lengths of rope or plastics material have also been found to be effective.

However there is generally no simple solution available. Additional damping alone is seldom sufficient. Where galloping is predicted or has been observed specialist advice should be sought.

9.9 Corrosion protection

The use of the largest practical wire diameters will generally facilitate application to the rope of adequate corrosion protection.

All steel wire ropes should be manufactured from galvanized round steel wire which should conform to BS 443 or BS 2763.

Guy ropes made up from galvanized steel wires should be further protected by a layer of grease or appropriate barrier. Care should be taken to ensure that the protective layer is compatible with the lubricant used in the manufacture of the ropes.
Sheathed steel ropes protected by polypropylene impregnation do not need further protection unless the sheath is damaged during erection and use, but special treatment is necessary at terminations where the impregnation has been removed.
Non-impregnated sheathed ropes should not be used because of the risk of corrosion taking place undetected.

9.10 Insulators

The choice of insulators will depend on electrical and mechanical requirements.

The minimum ultimate strength should be guaranteed by the manufacturer and, if necessary, confirmed by destruction or proof tests on specimens.

Each guy insulator fitting should be designed so that even if an insulator suffers electrical failure the stability of the mast is still ensured.

Arcing arrangements should be made so that arcing does not occur along the surface of the insulating materials adjacent to the steel fitting.

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

Annex A (informative)
Design of battens

The thickness of plate battens should be not less than 1/50 of the minimum distances between welds or fasteners. The slenderness of sections used as battens should not exceed 180. The width of an end batten along the axis of the main components should be not less than the distance between centroids of the main members and not less than half this distance for intermediate battens. Furthermore the width of any batten should be not less than twice the width of the narrower main component.

The battens and the connections between them and the main components should be designed to carry the main forces and moments induced by a transverse shear at any point in the length of a member equal to not less than 2.5 % of the maximum axial load in the member. For members carrying bending stresses calculated from eccentricity of loading, applied end moments or lateral loads, the battens should be proportioned to resist any shear due to bending in addition to the above mentioned value of not less than 2.5 %.

Battens should be connected to the backs of angles parallel to both the rectangular axes of the members. They should alternate in each plane and the effective length of a main component should be taken as the spacing centre-to-centre of the battens in the same plane.

The transverse shear of not less than 2.5 % of the axial load should be taken as acting perpendicular to the minor axis of the member. The battens in each plane should be designed for the components of this shear resolved perpendicular to the rectangular axes plus any transverse shear due to the weight or wind resistance of the member.

Information on other compound configurations can be obtained from BS 5950-1:1990, **4.7**.

Annex B (informative)
Terminations commonly used for guys

B.1 Guy sockets (efficiency 95 % to 100 %)

This method is particularly suitable for terminating spiral and locked coil strands. It involves fitting the brushed end of the strand into a tapered bore and filling with either white metal, zinc or synthetic resin.

B.2 Loops

The following methods of termination are most applicable to multi- stranded ropes. The end of the rope is passed round the groove in the thimble and fastened back to itself by one of the following methods.

a) *Hand splicing (efficiency 60 % to 90 %).* This is the traditional method of forming a terminal loop.

b) *Ferrule secured eye termination (FSET) (efficiency 90 % to 95 %).* The ferrule is placed over the looped back end and body of the rope and then swage pressed. This process is covered by BS 5281.

c) *Rope clamps (efficiency 20% to 90%).* Bulldog grips come into this category and permit relatively easy site termination but they are generally not advisable for permanent new masts and require re-tightening at regular intervals.

Wedge and socket anchorages are covered by BS 7166.

B.3 Factory formed helical tension terminations (efficiency 90 % to 100 %)

These are preformed strands that have been previously coated by some substance (such as abrasive grit), which are wrapped around the periphery of the wire rope for a length of up to 2 m, dependent on guy size and type. The preformed strand therefore provides a loop termination on site which can be adjusted. It is necessary to provide a rigid thimble to maintain the loop diameter at the connection.

B.4 General

Other methods can be used for terminating guy ropes provided their efficiency and durability can be demonstrated.

Provided the efficiency of the termination is not less than 90%, it may be assumed to have the same ultimate strength as the guy rope. No extra allowance need be made to the material factor γ_m to be applied to all guys and linkage components.

The size range of ropes on which the above terminations can be used is as follows.

a) Hand splicing — full range;

b) Loop produced by FSET — full range;

c) Socket — standard — 8 mm to 41 mm diameter;
— special — 32 mm upwards;

d) Preformed grips, maximum — 25mm diameter.

For guidance only, typical efficiencies are given in Table B.1.

Table B.1 — Static tensile efficiencies of terminations

Type of termination		Efficiency in tension[a]		
		Wire rope	Spiral strand	Locked coil
Hand splicing		60/90 %	NA[b]	NA
Loop by FSET		90/95 %	90 %[c]	NA
Swage pressed		95 %	95 %[d]	NA
Sockets	Standard	100 %	95 %[e]	NA
	Special	100 %	100 %	100 %
	Wedge	60/90 %[f]	NA	NA
Rope clamps		20/90 %	NA	NA
Factory formed helical tension terminations		100 %	100%[g]	NA

[a] Efficiencies based on minimum breaking loads.

[b] NA indicates termination not applicable to the product.

[c] More than one ferrule may be needed to achieve this efficiency, dependent on strand size.

[d] Dependent on strand construction and length of pressed shank.

[e] Increased size of standard socket will be needed.

[f] Dependent on number of grips fitted and torque applied to fastenings.

[g] Limited to the maximum size of 25mm diameter strand, and dependent on the length of grip applied.

Annex C (informative)

Erection tolerances for guyed masts

Final plumbing and guy tensioning should be done in surface wind speeds not greater than 5 m/s (10m/s gust speed). If higher wind speeds are experienced during this operation, calculations should be done in order to properly compensate for the effect of the wind.

Final plumbing and guy tensioning normally proceeds from the lowest guy level upward. The final position of the centroid of the mast column at each guy attachment should be such that the maximum initial slope of the mast column in still air between adjacent attachments or between any similar interval of height is less than the following:

— 0.05 % for masts of height > 150 m;

— 0.10 % for masts of height ≤ 150m.

For masts not subject to lateral loading in still air the resultant horizontal component of the initial tension of each of the guys at a given level should not exceed 3 % of the average horizontal component of the initial 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.

Where a mast is subject to lateral loading in still air, special consideration should be given to the selection of initial guy tensions. In such cases it is always preferable to ensure that the mast column is vertical, or inclined but straight, within the tolerances noted above.

Where non-plumb masts are being considered or where take-offs between guy points are being used great care needs to be taken to ensure that the designer's assumptions are being realized and that all of the designer's requirements are recorded and can be interpreted and measured on site.

The torsional deflection of the mast column in still air should be less than the following:

— 0.05° per 3 m and 1.5°overall for masts of height > 150 m;

— 0.10° per 3 m and 2.0°overall for masts of height ≤ 150m.

In addition, the torsional deflection between adjacent guy attachments should be less than 1.0°.

Bibliography

Standards publications

BS 5281, *Specification for ferrule-secured eye terminations for wire ropes.*
BS 7166, *Specification for wedge and socket anchorages for wire ropes.*

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